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Progressive Collapse Behavior of Reinforced Concrete Structures with Deficient Details

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Progressive Collapse Behavior of Reinforced Concrete Structures with Deficient Details

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Dedicated to My Beloved Wife, Parents, and Sister

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Progressive Collapse Behavior of Reinforced Concrete Structures with Deficient Details

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Damage from abnormal loading such as explosion, bombing, and sudden external impacts on elements of a structure can range from a loss of individual elements to total collapse. Progressive collapse has been a concern for many years, but recent acts of terrorism including the destruction of the World Trade Center and major damage to the Pentagon have renewed demand for methods to improve behavior of structures under these abnormal events. Progressive collapse can be defined as damage disproportional to the triggering mechanism. Design of structures against progressive collapse has not been an integral part of structural design. However, some codes such the GSA and UFC guideline have detailing requirements to reduce the likelihood of progressive collapse. It is difficult to predict the manner in which progressive collapse will propagate because the nature of loadings or triggering events are not well defined, and behavior of structural elements during a progressive collapse is not understood. In this study, three-dimensional nonlinear static and dynamic analyses of structures prone to progressive collapse are

conducted using commercially available programs. The analysis is assumed to be independent of the cause of damage. The initial objective is to simulate structural behavior when load-carrying members are removed under the effects of abnormal loadings. After the critical member is removed, redistribution of the forces to other elements is investigated. In addition, the capability of resisting redistributed loads is examined to determine if adjacent elements participate in producing a progressive collapse.

Dynamic effects due to removal of a critical column suggested by the GSA and UFC guidelines are compared with analytical results. The response of a robust structure is compared with the response of a structure with deficient details. When a critical column is removed, structural details related to connections, insufficient transverse reinforcement for shear in beams, and lap splices in columns are studied to demonstrate how components may be affected by removal of critical elements.

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CHAPTER 1

Introduction

1.1 BACKGROUND

Since the World Trade Tower collapsed after a terrorist attack, great interest in progressive collapse behavior of structures has been shown by the research and design communities in the US. Progressive collapse can be defined as a chain reaction of structural failure extending from local damage to collapse or extensive damage to a large area of a structure. Significant collapses such as the Ronan Point apartments [55], Alfred P. Murrah Federal Building [19], and World Trade Center [60] had a profound impact on society and on the engineering profession. These collapses initiated many discussions and suggestions for changes in code provisions.

Studies of progressive collapse have ranged from simulation of prototype structures to testing of structural elements. Analytical studies conducted by research groups in corporations or institutes have examined progressive collapse of two- and three-dimensional frame structures [10, 42, 59, 63]. A number of analyses have dealt with identifying the weakest link in steel and concrete frame structures. Experiments of strengthened members, such as a concrete column wrapped with carbon fiber polymer and concrete walls or masonry walls strengthened by fiber sheets, have been conducted [20]. Strengthened components in structures are tested because it is important to confirm that there is sufficient resistance of elements vulnerable to an external threat. If sufficient external energy can be absorbed after a severe explosion or impact, a robust or strengthened structure can resist extreme events.

Detailing deficiencies may limit the development of alternate load paths to distribute unexpected loads that were not considered during design and construction. If severe deficiencies are found, it is advisable to strengthen the deficient components and provide sufficient ductility to increase load redistribution capacity. Retrofit schemes to make a robust element should effectively prevent a structure from collapsing. Steel plates are often used to strengthen connection areas in steel frame structures [20]. For a reinforced concrete structure, specific local resistance in deficient components may be improved by wrapping with composite materials that have already been used in many engineering fields [20]. Four cases of progressive collapse are demonstrated as follows:

- Ronan Point Apartment Building [55, 57]

The Ronan Point apartment building shown in Fig 1.1 collapsed in 1968 due to a gas explosion on the 18th floor. This gas explosion blew out the load-bearing precast panel on the 18^{th} floor, causing a chain reaction collapse above the floor and all the way down to the ground. Four residents were killed, and corner panels of the apartment were demolished. Walls of the collapsed building were unreinforced so that only bond and friction resisted all forces by gravity loads [57]. Therefore, the apartment building had lack of continuity between members and limited system ductility. The British code provisions did not have a specific regulation for structural response under progressive collapse at the time of collapse [55]. This event triggered research and led to code modifications.

- Alfred P. Murrah Building [18, 19]

The Alfred P. Murrah building in Oklahoma City in 1995 showed progressive collapse by a truck bombing attack at the first floor. This truck bomb failed three columns at the first floor in that building [18]. The loss of three columns led to failure of transfer girders, causing collapse of the girders and floors supported by these columns. The structure had insufficient alternate load paths to absorb unexpected loads, leading to the progressive collapse shown in Fig 1.2.

- World Trade Tower [22]

Planes were flown into the main WTC towers on 11 September, 2001, causing the entire collapse of the two towers. Although the WTC towers collapsed due to effects of fire, failures associated with progressive collapse in buildings are shown in Fig 1.3. Sufficient redundancy helped the structure to sustain the damage for about one hour until the effects of intense fires caused columns to fail. When the remaining load-bearing elements of the building could not sustain loads, the entire collapse of the building occurred with the loss of more than 3000 people [22].

In a tall building like one of the WTC towers, the consequence of local failure may be catastrophic.

- Sampoong Department Store [40]

On 29 June, 1995, the Sampoong department store located in Korea collapsed entirely due to loss of load carrying capacity of the roof, producing a pancake collapsing behavior shown in Fig 1.4. This collapse caused the death of more than 500 people and the entire loss of the building. The building owner changed structural plans from the original design to install escalators and to add air conditioning units on the roof. The roof was not designed for large mechanical equipment and collapsed on the floor below, causing progressive collapse of the entire building [40].

Figure 1.1 Ronan Point apartment collapse [55]

Figure 1.2 Murrah building collapse
(http://en.wikipedia.org/wiki/Oklahoma_City_bombing)

Figure 1.3 WTC tower collapse
(http://www.solcomhouse.com/Worldtradecenter.htm)

Figure 1.4 Sampoong department store collapse [40]

Collapses of structures reported in the US and Canada between 1962 and 1971 are shown in Table 1.1, and 15 to 20% of all collapses involved progressive collapse [62]. In addition, collapses of 225 structures including apartment and office buildings were reported in the United States between 1989 and 2000 [64]. Notably, 54% of those 225 building collapses occurred between 1998 and 2000 [64]. Various causes such as natural hazards, maintenance mistakes, and construction errors were involved. It was difficult to distinguish if progressive collapses contributed due to limited information about the failures.

Table 1.1 Relative frequency of progressive collapse [62]

Year	Total collapse	Progressive collapse	% of failures involving progressive collapse
USA (1968-1971)	110	22	20%
Canada (1962-1971)	495	72	15%

1.2 RESEARCH OBJECTIVES

A significant portion of the reported collapses include progressive collapse that often leads to large human and property losses. In order to reduce the potential of progressive collapse, detailed behavior of a structural system is needed when a structural member is damaged.

Responses of concrete frame structures and flat plate structures will be examined by conducting nonlinear static and dynamic analyses. Dynamic effects of removal of a vertical load-carrying element will be examined for a robust structure and deficient structure. Alternate load path, that is load-redistribution to the adjacent members, and specific local resistance, that is a strengthening a member for a certain threat, will be explained.

This study illustrates methods to prevent instantaneous loss of elements and explains how to establish alternate load paths to lead to load-redistribution when vertical members are severely damaged and lose load-carrying capacity.

1.3 SIGNIFICANCE OF RESEARCH

Very few studies of progressive collapse of reinforced concrete structures have been reported. Many analytical approaches involve two- and three-dimensional analyses. In this study, an analytical approach focused on simulating structural response of three-dimensional structures was considered. Many guidelines allow for linear procedures for designing against progressive collapse. In addition, there have been very few studies for deficient existing structures susceptible to progressive collapse at extreme events. In this study, the emphasis is on capturing realistic structural response of three-dimensional structures using nonlinear static and dynamic analyses. Deficient aspects in existing concrete structures are investigated for vulnerability of a deficient structure to progressive collapse.

1.4 REVIEW OF LITERATURE (PERTINENT TO THIS STUDY)

The Ronan Point apartment in 1968 collapse provoked active research in the engineering community in Europe and US for better understanding of progressive collapse [14]. Investigations were conducted to find errors in design and construction procedures, but the collapsed structure was designed using the code provisions in place at the time. Although warning about progressive collapse in a structure was issued prior to the Ronan Point apartment collapse, extensive research related to progressive collapse started after the Ronan Point event.

Section 1.4.1 is a summary of design consideration and risk analyses related to progressive collapse. Development of analysis procedures will be discussed in section 1.4.2, and experimental studies will be discussed in section 1.4.3. Code provisions to prevent progressive collapse are excluded here but will be discussed in Chapter 2.

1.4.1 Design consideration and risk analyses

1968~1975

The initial trigger to study a structure vulnerable to disproportionate collapse was the Ronan Point apartment collapse in 1968. After the disaster of the Ronan Point apartment, the British design code introduced methods for designing a structure specifically for an abnormal load. The National Building Code of Canada included regulations regarding progressive collapse in 1975 [21].

Several workshops related to progressive collapse were held in US after the Ronan Point Collapse. In a workshop in 1975, Breen [13] reviewed new regulations established in England after the collapse of the portion of the Ronan Point apartments and development of US provisions at the time. The study focused on detailed design configurations of precast concrete structures against progressive collapse. Because connection regions of precast concrete structures were vulnerable to failure for an abnormal load, Popoff [57] also illustrated weak points of connection regions and retrofit methods to prevent progressive collapse. Taylor [62] emphasized integrity in a structure to reduce the risk of collapse during construction. He discussed the role of ductility, specific local resistance, and alternate load paths in resisting progressive collapse.

1976~1994

Because it was difficult to determine specific abnormal loads for design, load factors for an abnormal load and failure rate of a structure were often calculated by probabilistic and reliability approaches. Ellingwood and Leyendecker [24, 27] illustrated probabilistic approaches to determine abnormal loads for designing structures. Probability of collapse was calculated when dead, live, and wind loads were considered. Load criteria for abnormal loads similar to the British code were suggested.

Because many progressive collapses in structures were observed during construction [62, 64], Monsted [51] and Webster [65] studied events of progressive collapse during construction of a structure. Monsted [51] emphasized that alternate load paths and catenary action had to be provided when a load bearing element was damaged. He specifically demonstrated how a primary component such as a load bearing wall and joint detailing could be taken into account to provide resistance against progressive collapse. Webster [65] studied excessive loading on a slab during construction and found that loads could often exceed design service loads, leading to punching shear failure and a chain reaction of a structural failure elsewhere in the structure. Reliability concepts were adopted to obtain the probability of survival of a structure under punching shear failure. Bennett [11] studied reliability of a panel structure similar to the Ronan Point apartment. He focused on the sequential nature of progressive collapse in determining probability of failure of the structure. Strengthening of a structural system could be conducted for most critical portions of a structure.

Consideration of progressive collapse in design generated a number of controversial terms and debates among engineers in 1970s in the US. Breen and Siess [12] tried to improve dialogue among researchers by illustrating issues related to progressive collapse. The study also focused on establishing safety standards in the engineering field and trying to solve difficulties in designing against progressive collapse.

1995~2000

After the Oklahoma City bombing [19, 58], it was clear that the US was not immune to a bombing attack. Prendergast [58] suggested that a standoff distance, which can be provided by barriers or walls to prevent access of external threats and structural redundancy, could significantly reduce blast peak pressures. Cantilever behavior or catenary action of the adjacent, undamaged elements near damaged areas was discussed to reduce loss of life and damage in properties.

Erling [29] studied progressive collapse loads in a flat roof structure. In previous research [24], load combinations including dead, live, and wind loads for alternate load paths against progressive collapse were considered. Erling considered that progressive collapse in a structure was more prone to abnormal loads such as snow loads. Instead of considering live loads in load combinations, he discussed how snow loads can be added to dead and wind loads and be applied in load combinations for design against progressive collapse in flat-roof structures.

Recent development

Linear static, nonlinear static, linear dynamic, and nonlinear dynamic analyses can be used to evaluate structural response. Structural behavior under seismic forces has often been evaluated using these analyses. Marjanishvili [47] reviewed advantages and disadvantages of conducting each analysis and suggested that each analysis procedure should be checked and selected for a target structure in a progressive way from linear static to nonlinear dynamic analysis.

Since the General Services Administration (GSA) guidelines [7] were released, many engineers have used the guidelines to check safety of a structure against progressive collapse in the preliminary design process or during design. Commercial structural analysis programs have often been used. Performance of three-dimensional buildings to resist progressive collapse was studied by Baldridge and Humay [10]. The inherent ability of RC beam-column frames designed for seismic effects to withstand abnormal loading was demonstrated. Analyses were performed using ETABS [30] following the GSA guidelines.

Linear static analysis considering demand capacity ratio (DCR) confirmed that structures with seismic detailing performed well when a key element was suddenly removed. Robustness and alternate load paths were verified from removal of the critical column at different locations.

Multihazard Mitigation Council Workshop (2002)

After the WTC tower collapse in 2001, the Multihazard Mitigation Council (MMC) organized a workshop in 2002 to present research regarding progressive collapse in the US. This workshop was conducted in cooperation with the GSA. The GSA guidelines used in this dissertation were presented at the workshop.

Burns [28] illustrated the GSA requirements and appropriate retrofit methodologies for Federal government buildings and historic government buildings. He introduced general code provisions to guide design criteria for commercial buildings and to prevent progressive collapse in high-rise buildings. Cagley [15] illustrated the importance of a prescriptive design code for abnormal loading conditions. He stressed the use of performance based design in the guidelines.

Due to uncertainty of abnormal loads and cost of rehabilitation, it is difficult to predict design loads and corresponding retrofit schemes to prevent progressive collapse. Krauthammer et al. [44] recommended that abnormal loads should be investigated to prevent progressive collapse. They discussed the limitation of the analysis techniques and programs to take into account nonlinear behavior of structures. An analytical technique for progressive collapse analysis was needed because there were very few numerical analysis techniques for studying damaged structures.

A study by Crawford [20] differentiated between several potential abnormal loads such as thermal loads, impact loads, and blast loads. He pointed out that most design criteria were based on flexural response with limits on rotational capacity and ductility and did not address other modes such as shear failure or buckling. Crawford criticized the current design paradigm using a simplified analysis that could provide reasonable accuracy for extreme events. For better analytical results, analysis tools such as FLEX, LARSA, ABAQUS, and ANSYS [60] were recommended for complicated structures. In this dissertation, Perform-Collapse [56] developed by Dr. Powell and distributed by RAM International was used to conduct nonlinear analyses.

A probabilistic approach for progressive collapse was proposed by Ellingwood at the workshop in 2002 [26]. His studies explained load and resistance factor design criteria for progressive collapse and dealt with the structural capability for resisting damage without progressive collapse in a probabilistic and reliability approach.

Progressive collapse of the Alfred P. Murrah Federal Building was revisited by Corley [18, 19] in a detailed investigation. The study provided more details of the building and illustrated the effects of seismic detailing of special moment frames for preventing progressive collapse. From the analyses that he conducted, the structural integrity requirements and mechanical butt splices required in the current ACI code could have prevented the failure of the building or reduced the catastrophe. Additionally, other structures located near the Murrah building were also inspected to check the amount of damage in each structure [19]. As one of the retrofit options to reduce a structural failure, a compartmentalization system was recommended to provide alternate load paths.

Code provisions developed in Canada and United Kingdom were reviewed and studied by Dusenberry [21] and Moore [52] at the workshop. Moore [52] discussed the significant effort in the UK to prevent progressive collapse. Structures that performed well under bombing attacks were described to show the effectiveness of code regulations for abnormal loads.

1.4.2 Analytical approaches to Progressive Collapse

Some analytical tools to investigate structural response involving progressive collapse were discussed in Reference 36. Several analytical studies for progressive collapse were conducted for simple structures [11, 38] to validate analytical procedures and focus on obtaining fundamental aspects of the progressive collapse behavior. Progressive collapse resistant design in steel frame structures was studied by Gross and McGuire [38]. A two-dimensional steel frame was analyzed for various load combinations, including wind effects. Debris load effects from upper floor were also considered. Steel frame structures were found to be the least prone to progressive collapse due to inherent ductility of steels and continuity of bolted and welded connections. Gilmour and Virdi [36] also developed a computer program for planar steel and concrete frames, including effects of local damage, alternative load path, and debris loads.

In references [11, 36, 38], analytical procedures for progressive collapse of RC structures were not studied, but the lack of studies related to reinforced concrete structures was noted and stated that analytical procedures for concrete structures should be developed.

Kaewkulchai and Williamson [42, 43] emphasized the importance of dynamic effects in a structure experiencing progressive collapse. The authors developed a computer program using a damage index to evaluate member failure. The study concluded that dynamically spreading effects of the response should be taken into account in analyzing a structure under abnormal loading resulting in partial or global collapse. Dynamic analysis results generated more hinge development and larger deflection than static analysis results. The studies focused on steel frame structures and did not provide analytical approaches for RC structures.

A computational algorithm for a two-dimensional steel frame was developed to simulate structural behavior when critical elements were eliminated [16]. Initial removal and indirect removal of a critical element were considered. Indirectly removed elements were considered when an element no longer resisted the applied loads. Progressive collapse related to local buckling was investigated. This study demonstrated a steel frame structure with local buckling showed much faster failure initiating time than a structure without local buckling. Therefore, consideration of local buckling was recommended for study of progressive collapse. However, this study also did not include analytical procedures and results for RC structures.

When an abnormal load is applied and causes a floor to fail, it is very difficult to define the damaged area to simulate dynamic impact of loads. A computational code for considering dynamic effects from upper floors was developed by Grierson et al. [37]. The consequences of shear failure of the floors caused by dynamic debris loads were included in the analyses. The study emphasized that shear failure caused by the debris loads was a significant failure mechanism for progressive collapse. Due to complex characteristics of simulating impact loads from upper floors, a simplified method was suggested in a recent study by Vlassis et al. [63]. The simplified program was based on a simply supported beam. Hinge rotational capacity and axial tension capacity at a connection was studied to reflect the influence of ductility on response. This study provided over-conservative analytical results to represent rotational behavior at connection region.

Due to difficulty and complexity of designing against progressive collapse, simple and quick processes to check for the potential of progressive collapse are required. However, it is not easy to make a decision regarding the vulnerability of a global structure to progressive collapse. A recent study [17] attempted to solve this difficulty in a design. Multiple simulations for a simple structure were conducted to find the possibility of a failure in the components of structures. This study indicated that more detailed tools were needed to capture more accurate progressive collapse behavior.

Recent studies of three-dimensional structures were conducted regarding progressive collapse [10, 59]. Linear static analysis using ETABS for a three-dimensional concrete structure was conducted following the GSA guidelines [10]. The study [10] demonstrated RC structures designed for seismic forces could distribute abnormal loads and resist progressive collapse. Nonlinear static and dynamic analyses conducted for a three-dimensional steel frame structure [59] demonstrated that the dynamic amplification factor provided in the GSA guideline, representing dynamic effects when an abnormal load was applied and badly damaged a column, was conservative.

Although various analytical research were studied, most studies have focused on two- and three-dimensional steel frame structures. Only a few studies of RC structures have been conducted [10, 46]. However, they did not involve three-dimensional modeling of concrete structures with slabs using nonlinear static and dynamic analyses.

1.4.3 Experimental Studies related to Progressive Collapse

Progressive collapse is often caused by abnormal loads that are not considered during the design process. Alternate load paths for excessive loads simulating abnormal loads to prevent progressive collapse were conducted by Astaneh [9]. Exterior continuous steel cables along members were suggested to provide catenary action in beams when a column has lost its load-carrying capacity. After a center column was initially removed, behavior of the floor slab with continuous steel cables was compared with a structure without the cables, and results showed that floor slabs of steel structures with steel cables can be prevented from progressive collapse.

The incorporation of earthquake resistant detailing in a concrete structure can reduce the possibility of progressive collapse [33]. However, the application of seismic strengthening to address progressive collapse prevention has not been studied experimentally.

Most experiments conducted on concrete structures have focused on the effects of load reversal, loading capacity against shear failure, and general lateral capacity. Progressive collapse response is greatly influenced by the gravity loads after local failure occurs. Although shear failure is most common for columns subjected to seismic forces, most experiments were terminated when shear failure in a column occurred. However, there may be residual axial load capacity to sustain gravity loads or by redistribution of loads to adjacent beams or slabs. A few researchers have investigated residual capacity through load redistribution after damage of critical elements has occurred.

Gravity load collapse of a reinforced concrete frame was studied by Elwood and Moehle [28, 49]. Their study found residual axial capacity could prevent collapse of a structure although shear failure in a concrete column had occurred. A formula using a shear-friction model was suggested to simulate additional axial load capacity after shear capacity was exhausted [28, 49, 50]. The study [28] also considers residual capacity of adjacent elements in analyses after a component fails and leads to redistribution of the applied loads.

Flat plate structures are one type of structure most prone to progressive collapse [39]. Punching shear failure in a flat plate structure was often observed even before yielding of the bottom reinforcement of slabs occurred [39, 48]. Hawkins and Mitchell [39] found that punching shear failure at exterior columns had a low possibility of leading to progressive collapse. However, unless continuous bottom reinforcement through a column or good anchorage was provided, tension membrane by slabs was not effective and could lead to catastrophic failure. Proper detailing of slab reinforcement at a column support enabled a damaged slab to hang from its support. Therefore, well detailed flat slab structures were capable of resisting additional loads even after punching failure at a support region occurred.

Due to recent terrorist attacks, realistic behavior of concrete structures under blast loadings is needed. One of the studies related to blast mitigation is being conducted at the University of California at San Diego [53]. A device that simulates blast effects has been used to study behavior of concrete structural components. A full-scale concrete structure was tested under impact loads supported by the Department of Defense. However, many researchers argue that impact and blast simulation is not realistic and can not simulate strain-rate related issues that the blast loads contain. Therefore, a few real blast experiments were conducted for RC retrofitted structures [20, 46]. These retrofitted structures were able to resist the blast loads well. A study by Malvar [46] reported behavior under blast load and suggested appropriate retrofit schemes. Both specific local resistance and alternate load paths were considered to rehabilitate the structure. Although the suggested retrofit schemes are not necessarily applicable to all concrete structures, the fundamental retrofit concepts to prevent progressive collapse were defined. The study recommended that exterior frames can be rehabilitated by providing specific local resistance using steel jacketing or wrapping with fiber material. Interior frames can be supported by developing load paths using adjacent components.

1.5 OUTLINE OF THE DISSERTATION

This dissertation is organized as follows:

In Chapter 2, code provisions related to progressive collapse and general concepts to reduce progressive collapse are introduced. Analysis procedures used in this study are discussed in Chapter 3. Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses will be discussed. In Chapter 4, verification of the computer program is conducted. Analytical results will be compared with experimental data from various slab models. Dynamic analytical results will also be compared with results reported in the literature. In Chapter 5, a robust structure is analyzed. Linear and nonlinear analyses are conducted to compare consideration of the dynamic effects with the GSA guidelines. In Chapter 6, deficient aspects in existing structures are defined and discussed. Commonly used retrofit schemes are introduced. In Chapter 7, analysis results and discussions are provided for a deficient reinforced concrete frame structure and a flat plate structure. In Chapter 8, summary, conclusions and recommendations for future research are provided.

CHAPTER 2

Code Provisions and General Concepts to Reduce the Potential for Progressive Collapse

General concepts for reducing the possibility of the progressive collapse of structures are discussed, and provisions in various codes related to structural integrity and progressive collapse are introduced.

2.1 General Concepts for Reducing Progressive Collapse

2.1.1 Event control

It is almost impossible to control a specific threat before it occurs because such threats (fire, aircraft impact, gas or bomb explosion, and vehicular collision) are arbitrary events. Furthermore, it would be difficult for a structural engineer to analyze the magnitude of specific threats. Standoff distance, defined as the distance between the nearest structural component and the defended perimeter, is a good example of event control or reducing the impact of specific threats [7]. While the standoff distance can be taken into account for important facilities such as chemical factories, government buildings, and power plants, it may not be a solution for commercial buildings. It is very difficult for a building owner to set up sufficient standoff distance in an urban area. Event control is not considered in this study.

2.1.2 Indirect method

Indirect methods to reduce the possibility of progressive collapse can be defined in terms of structural integrity, layout of the structural components, and detailing of the structural members, including connections [60]. Analysis or calculation for structural behavior is not required in this method.

In general code provisions such as ACI 318-05 [1], structural integrity reinforcement is required to improve redundancy and ductility in structures. In addition, shear resistance should always exceed the flexural capacity to ensure ductile behavior. In order to ensure continuity in structural components, tie forces such as those illustrated in Fig 2.1 are needed [4]. Internal, vertical, and peripheral ties connecting structural components as shown in Fig 2.1 can help a structure to develop catenary action when an adjacent member is damaged. When one of the critical columns is damaged and loses its load-carrying capacity, connecting spans deflect until rotational capacity provided by the adjacent beams or slabs is exhausted. Then, tie forces in connecting elements play a significant role in sustaining the structure. This behavior is defined as “catenary action”.

Internal ties should be provided from one edge perimeter to the other edge perimeter. These internal ties can be provided by continuous beams. Vertical ties are provided by columns or walls from the highest to the lowest level. Peripheral ties represent continuous ties around plan geometry. Horizontal ties to external columns or walls and corner column ties are provided by adequate anchorage among elements.

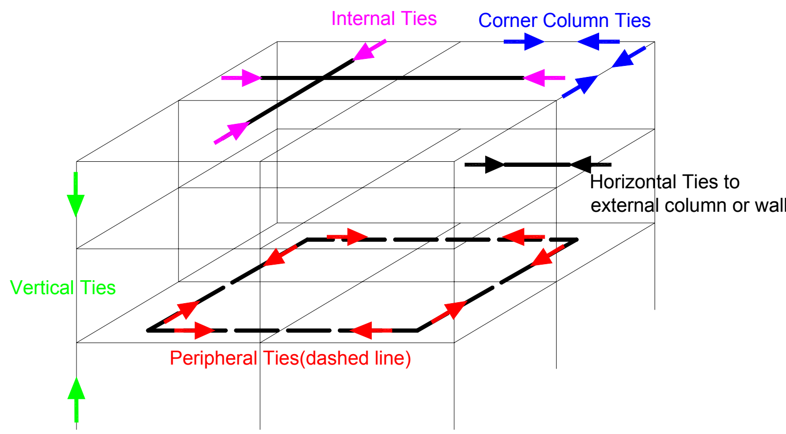


Figure 2.1 Tie forces defined in the UFC guideline [4]

2.1.3 Direct method

Alternate load paths and specific local resistance in structural members are considered in order to explicitly protect a structure from the results of local damage and its extent to global damage. Structural analyses using these methods include nonlinear static or dynamic analyses because structural members experience nonlinear behavior due to strains exceeding yield or even strain hardening in steels and wide cracking and crushing at the compression zones in concrete members.

2.1.3.1 Alternate load path method (AP)

When a damaged load-bearing element does not have the capacity to carry given gravity loads, adjacent members may help redistribute these loads as shown in the Figs 2.2-2.4. The load path before damage involves transfer of loads from the floor slabs to the beams and then to the columns. After damage in a column, load paths will be dramatically changed in the members adjacent to the damaged column. If adjacent members have sufficient capacity and ductility, the structural system will develop alternate load paths and allow occupants to evacuate. Alternative load paths must be considered for preventing progressive collapse because it is generally not practical to prevent all of the members from failing under unexpected loads.

For example in Fig 2.3, the floors will sag when the column is removed. The spans across the missing column could be supported by catenary action. Alternatively, if there are a number of floors above the missing column, the loads on the floors could be transferred upward through tension in the columns above and the remaining structure would help transfer the load to adjacent, undamaged spans, as shown in Fig 2.4.

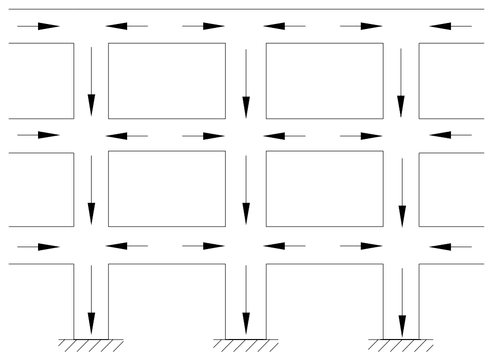


Figure 2.2 Flow of force before damage

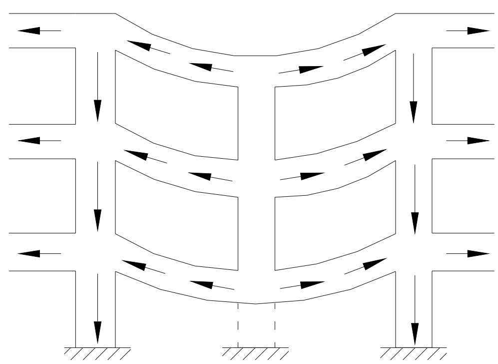


Figure 2.3 Flow of force after damage (catenary action)

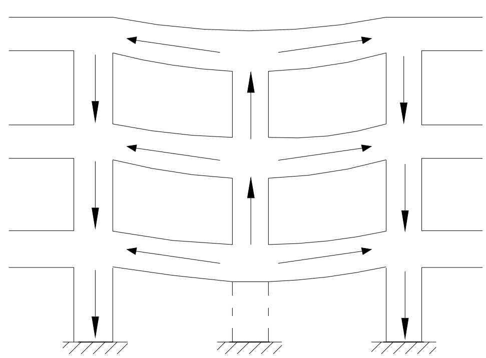


Figure 2.4 Flow of force after damage (tension in columns)

2.1.3.2 *Specific local resistance method (SLRM)*

One of the methods to make a structure robust against progressive collapse is to strengthen a specific key element in the structural configuration. A good example can be the structural elements designed to sustain a static pressure of 5 psi for gas explosions following provisions in the United Kingdom after the Ronan Point apartment collapsed [57]. A probability-based approach can be adopted to consider the exposure of the specific buildings to threats. Specific local resistance can also be applied to newly constructed buildings in design and to existing buildings for the retrofit schemes to resist extreme events. An abnormal load term may be included in a load combination below [26].

$$\text{Load} = (0.9 \text{ or } 1.2)\,(\text{Dead load}) + A_k + 0.5\,(\text{Live load}) + 0.2\,(\text{Wind load}) \qquad (2.1)$$

where A_k stands for the forces by abnormal loads such as explosion or collision of vehicles.

Some rehabilitation methods for specific local resistance are wrapping fiber-reinforced polymer (FRP) around the critical columns and installing fiber sheets under the beams or slabs [20]. Cost-effective retrofits should be focused on strengthening key elements.

2.2 CODE PROVISIONS

2.2.1 Great Britain

After the Ronan Point apartment collapsed due to a gas explosion, provisions regarding progressive collapse were added in 1970 in the Fifth Amendment of the British Building Regulation [57]. Buildings taller than five stories were required to meet requirements for damage areas after removal of one structural member or there were required to resist a specified pressure (5 psi) [57].

2.2.2 US

In 1971, the Department of Housing and Urban Development (HUD) initiated a study of progressive collapse in the United States. In the mid 1970's, HUD commissioned the Portland Cement Association (PCA) to develop standards for design criteria. The National Bureau of Standards continued studies of detailed design strategies for abnormal loads [57]. In 1972, the American National Standards Institute (ANSI) Standard A58.1 also introduced general remarks related to progressive collapse. After 1982, the American Society of Civil Engineers (ASCE) adopted the statement and has maintained it as part of ASCE 7. The current ASCE 7 contains indirect and direct design alternatives for the mitigation against progressive collapse. Structural integrity, such as good detailing and catenary action, was also emphasized to prevent progressive collapse. The New York City Code specified a tie force requirement and a damage limit as the lesser of 20% of the floor area or 1000 ft^2 in the horizontal direction for more than three story buildings.

Structural integrity provisions were introduced in the ACI code in 1989, and details for confining reinforcement in members have been included in Chapter 21 of ACI 318 for many years. Recent publications from the General Services Administration (GSA) [7] and the Unified Facility Criteria (UFC) [4] guidelines are introduced and compared in section 2.2.5.

2.2.3 Eurocode

The Eurocode [2, 3] considers progressive collapse as a rare event, and implicit methods are used to avoid damage disproportionate to the event. Tie forces (indirect method) are explicitly regulated. Alternate load paths are introduced, but the specific local resistance method is not addressed. This code recommends that linear static analysis can be conducted for mid-height buildings and nonlinear dynamic analysis can be conducted for buildings higher than 10 stories.

2.2.4 Others

The Canadian code adopted explicit design provisions against progressive collapse much earlier than the US. In the mid 1970s, the alternative load path and the specific local resistance for a key element were included. While tie force requirements as an indirect design method were not introduced, the importance of connection details for continuity between elements was emphasized to carry loads.

Swedish code provisions include an indirect design method (specified tie forces) and allow local damage in a structure by permitting large deformations and plastic behavior in the structure.

2.2.5 Comparison of GSA and UFC guidelines

Progressive Collapse Analysis and Design Guidelines [7] by the General Services Administration (GSA) and *Design of Buildings to Resist Progressive Collapse* [4] by the Unified Facilities Criteria (UFC) of the Department of Defense (DOD) have been widely referenced and accepted. Both codes have been developed to reduce progressive collapse in existing and new federal office buildings and military buildings when an abnormal loading condition is applied. The GSA document, issued in 2000, mainly focused on reinforced concrete structures. The 2003 version of the GSA document includes analysis and design requirements for steel structures [7].

Threat independent analysis and design are used in both the GSA and UFC code provisions because all possible threats such as blast, collision, and abnormal loads are difficult to take into account in terms of where these threats will be located and how large the impact will be on a structure. Instead, both codes suggest that sufficient internal energy dissipation be provided through good ductility, continuity of reinforcement, and redundancy of elements or load paths to reduce the progressive collapse potential of damaged structures.

The GSA document set specific categories to evaluate the potential for progressive collapse by using a flow chart summarized in Fig 2.5 which indicates whether further consideration of the resistance of a particular structure against progressive collapse is needed.

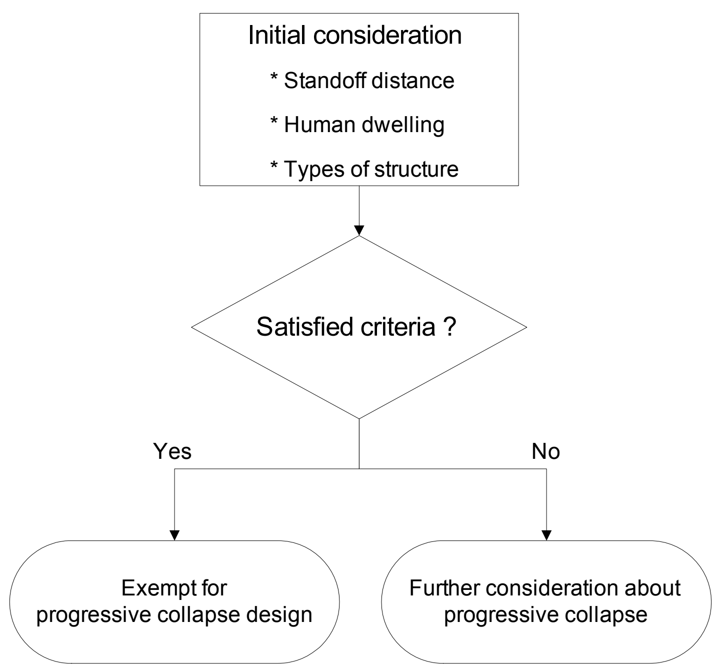


Figure 2.5 GSA design procedure for progressive collapse [7]

The UFC document specified four major categories (Fig 2.6) by differentiating among significance level of existing and new structures: Very Low Level of Protection (VLLOP), Low Level of Protection (LLOP), Medium Level of Protection (MLOP), and High Level of Protection (HLOP). Analysis and design strategies are determined based on these levels.

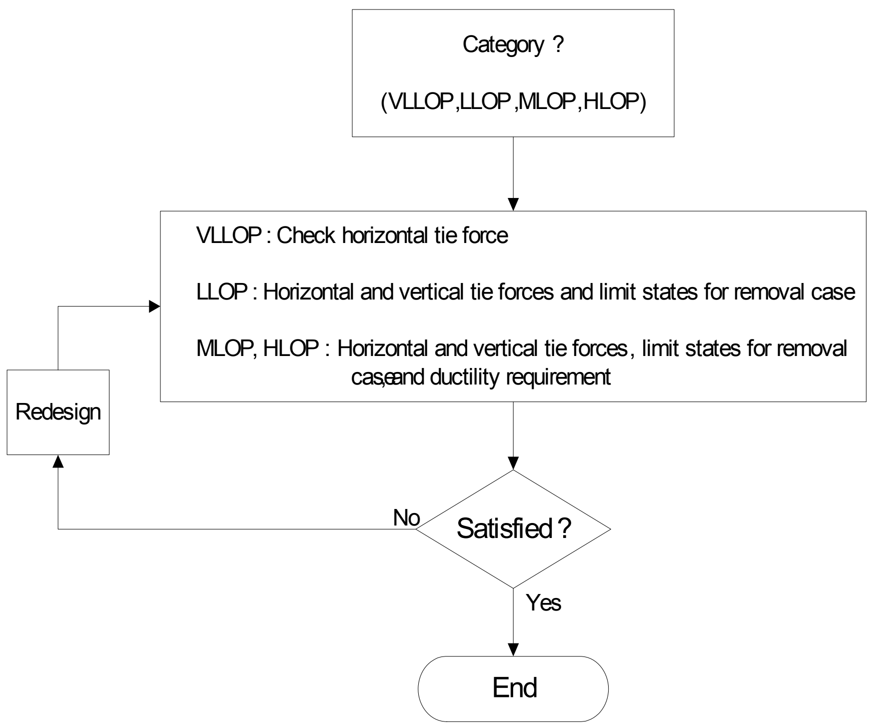


Figure 2.6 UFC design procedure for progressive collapse [4]

Although both the GSA and UFC criteria suggest that a nonlinear procedure for static and dynamic analyses be adopted in analyzing a sophisticated structure, the

GSA guideline deals with linear static and dynamic analysis procedures because simplified analysis can generate fast but conservative results. The guideline specifies that use of linear procedures should be limited to buildings with ten or fewer stories. Buildings more than ten stories and atypical structures (structures with irregularities) should be evaluated using nonlinear procedures. The UFC guideline describes linear static, nonlinear static, and nonlinear dynamic analysis procedures and provides some examples for defining hinge properties for flexural members for nonlinear analyses.

Both the GSA and UFC guidelines adopted notional removal of critical columns. A concept of notional removal of a critical column takes into account initial removal of a column when that column is badly damaged. Alternate load paths are considered in both of the guidelines. A strength increase factor of up to 25% as used in FEMA 273 [5] was included in concrete and steel material to reflect gain in concrete strength with time which leads to values higher than nominal design values. The increase for steel reflects strain hardening and actual strengths greater than nominal values. The GSA adopted a strength reduction factor of 1.0 as specified in FEMA 273/356 [5, 6] to represent strength of the existing structural elements while the UFC adopted the same reduction factor (ϕ factor) as the ACI code. Load combinations for the GSA and UFC guidelines show differences. The same dynamic factor of 2.0 is specified to simulate dynamic effects resulting from the removal of a load-bearing element. The load combination for the GSA guideline is used on all floors. For the UFC guideline, the amplified load is applied to the all stories above the removed element and limited to the bays supported by the removed element. The factors in the GSA and UFC documents are summarized in Table 2.1.

The GSA guideline indicates that possibility of progressive collapse is high when a Demand Capacity Ratio (DCR) exceeds 2 for typical structures (structures without irregularities) and 1.5 for atypical structures (structures with irregularities). DCR, defined as a ratio of acting force (demand) to ultimate, un-factored capacity, can be used when a linear static analysis is conducted. Moment-curvature from the equivalent elastic analysis should give the same area below the moment-curvature relationship from inelastic analysis. The concept of DCR is shown in Fig 2.7. The UFC guideline does not specify a DCR.

The load sequence is somewhat different when a critical column is removed. The GSA guideline considers initial removal of a column before any analysis is conducted. The UFC guideline suggests that an undamaged structure should be analyzed under gravity load before a critical member is removed. The damaged structure is analyzed to determine if specified limit states are reached. Damage limits for both guidelines are shown in Table 2.1. The UFC guideline for damage limit is more conservative.

Table 2.1 Factors used in the GSA and UFC

CATEGORY	GSA (Federal buildings)	UFC (Military buildings)
CAPACITY INCREASE FACTOR	Concrete: 1.25, Steel: 1.25	Concrete: 1.25, Steel: 1.25
DYNAMIC FACTOR	2.0 (Applied to whole system)	2.0 (Applied to all stories above the removed column)
STRENGTH REDUCTION FACTOR	1.0	ϕ (ACI code)
LOAD COMBINATION	Static analysis (all bays): 2(DL+0.25LL) Dynamic analysis: (DL+0.25LL)	Static analysis (not for all bays) : 2(1.2DL+0.5LL+0.2W) Dynamic analysis: (1.2DL+0.5LL+0.2W)
DEMAND CAPACITY RATIO (DCR)	DCR ≤ 2.0 for typical structure, DCR ≤ 1.5 for atypical structure	DCR is not used.
COLUMN REMOVAL	Analyze structure with column removed (notional removal)	Analyze structure with column in place and determine damage when column is removed
DAMAGE LIMIT	Exterior column removal: Smaller [Structural bays, 1800 ft^2] Interior column removal: Smaller [Structural bays, 3600 ft^2] [see 2.8 (a) and (b)]	Exterior column removal: Smaller [750 ft^2, 15% of total area] Interior column removal: Smaller [1500 ft^2, 30% of total area]

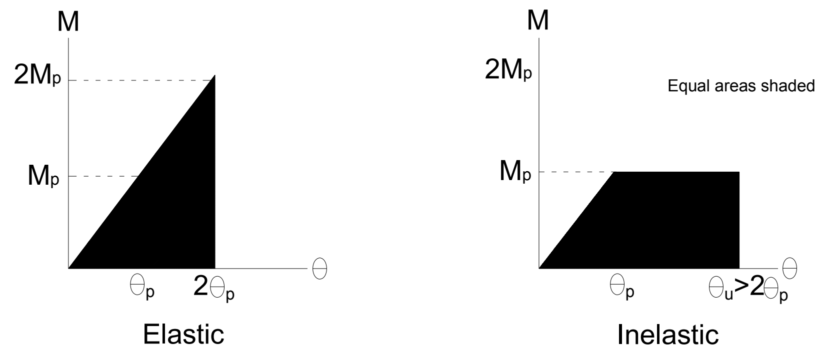
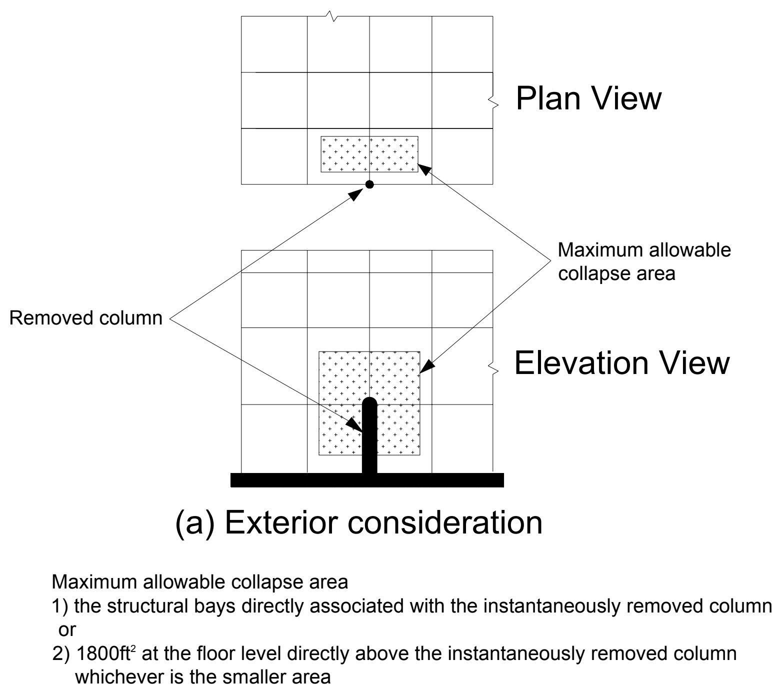


Figure 2.7 Demand Capacity Ratio [59]

The definition of damage area is somewhat different in the GSA and UFC documents. The GSA definition is illustrated in Fig 2.8 (a) and (b). The UFC definition is stated in section 3-2.6.1 of Reference 4.



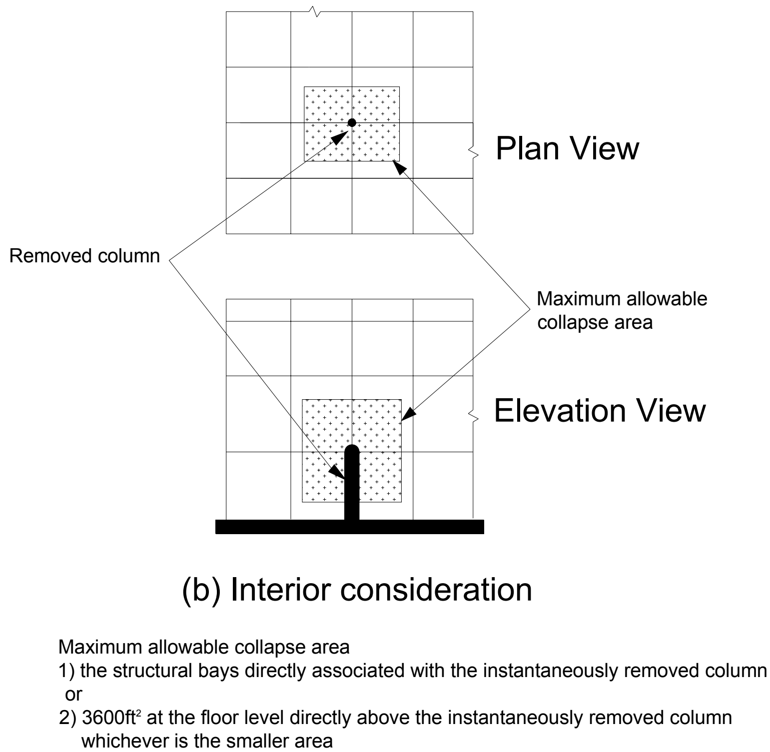


Figure 2.8 Limit of damage area [7]

2.2.5.1 Notional removal of the critical column and loading differences

Columns are notionally eliminated in Fig 2.9 to illustrate application of the dynamic factor for removing a critical column for the GSA or UFC. Removed columns are located at the corner, at the edge at a short and long side, and at the interior. External threats such as vehicle collision and blast can damage a column along the perimeter and internal explosions can deteriorate an interior column. Uncontrolled public access and underground parking places are also vulnerable.

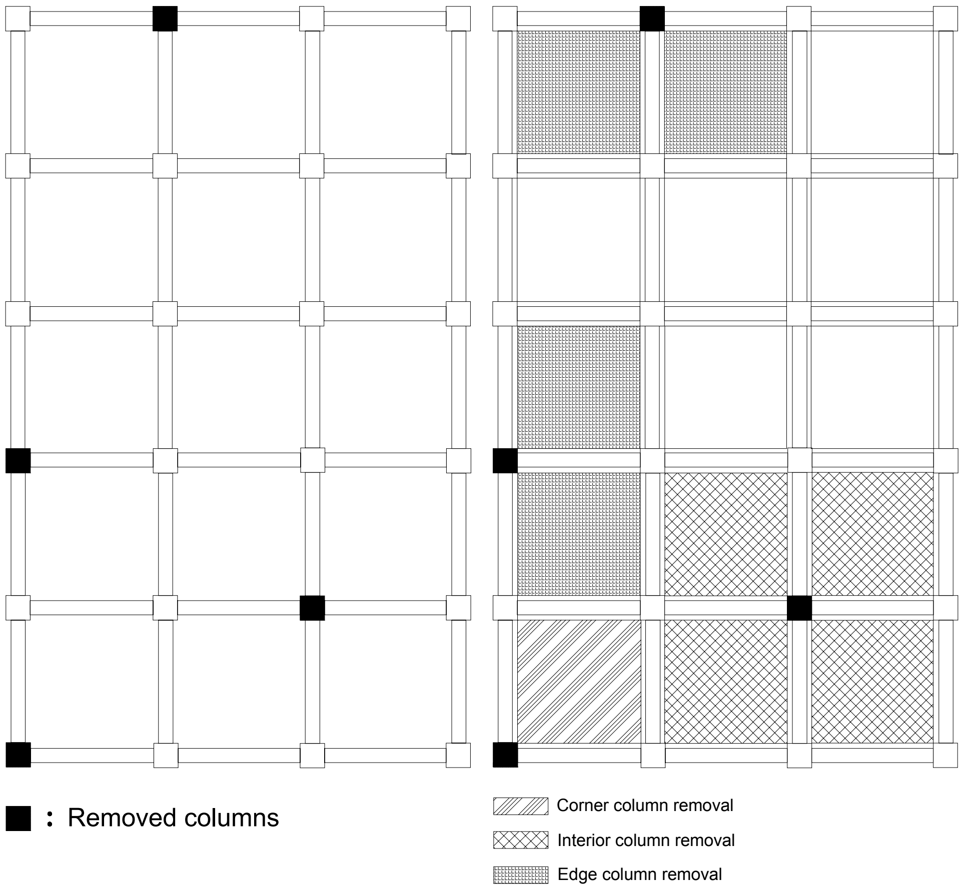


Figure 2.9 Plan view for the removed columns and areas affected (UFC)

Although both guidelines recommend the same dynamic factor (2.0), each guideline indicates the amplified areas in a different way. The GSA requires the use of a dynamic factor on the whole floor while the UFC dynamic factor is applied to the floor above the removed column locations as illustrated in the shaded areas of Fig 2.9. Removed columns are assumed to be located at the first story for the GSA guideline and columns at other stories are removed for the UFC guideline.

2.2.5.2 Acceptance criteria

The GSA and UFC guidelines are similar in some cases and different in the other applications of limit state criteria. Rotational limits in plastic hinge properties for nonlinear procedures for the GSA and UFC are shown in Table 2.2. The UFC guideline provides different values for low level of protection (LLOP) and medium and high level of protection (MLOP, HLOP). In the table, acceptance criteria for LLOP, MLOP, and HLOP are provided.

Reinforced concrete beams and slabs with tension membrane allow rotation up to a limit of 0.105 radian for the GSA and LLOP for the UFC guideline and 0.07 radian for MLOP and HLOP in the UFC guideline. Beams and slabs that are singly or doubly-reinforced flexurally without shear reinforcing allows up to 0.053 radian for LLOP and 0.035 radian for MLOP and HLOP in the UFC guideline. However, the GSA guideline does not differentiate between with or without shear reinforcing. Tension controlled RC columns in the GSA guideline allows plastic rotation angles up to 0.105 radian while the UFC guideline only provides a ductility ratio of 1.0 for columns. When the tension membrane effect is considered, both guidelines allow large plastic rotational angles.

Table 2.2 Comparison of Acceptance Criteria (GSA, UFC)

Component	GSA		UFC			
			LLOP		MLOP,HLOP	
	θ*	μ^+	θ*	μ^+	θ*	μ^+
RC beams and two-way slabs with shear reinforcing (**without** tension membrane)	0.105	-	0.105	-	0.07	-
RC beams and two-way slabs without shear reinforcing (**without** tension membrane)			0.053	-	0.035	-
RC beam and two way slab (**with** tension membrane) and L/h ≥ 5	0.21	-	0.35	-	0.21	-
RC column (tension control)	0.105	-	-	1.0	-	1.0
RC column (compression control)	-	1.0	-	1.0	-	1.0
Seismic column	-	-	-	3.0	-	2
RC frame	0.035	-	-	-	-	-

* Rotation (Radians)

\+ Ductility ($\mu = \dfrac{\varphi_u}{\varphi_y}$)

2.2.6 Summary

So far, general concepts of designing against progressive collapse and code provisions have been illustrated. Event control or standoff distance was not discussed.

Requirements reducing potential for progressive collapse have been updated in the GSA and UFC guidelines. The requirements are related to new Federal or military buildings. Moreover, the GSA guideline is primarily based on linear-static analysis. Nonlinear analysis procedures are recommended for complex structures. Although the UFC guideline shows how to establish hinge properties for a concrete structure, it is not sufficient to cover all cases for more accurate analyses.

CHAPTER 3

Analysis Procedure

3.1 TYPES OF ANALYSIS

Progressive collapse may be analyzed using linear static, linear dynamic, nonlinear static, or nonlinear dynamic procedures. Each analysis has advantages and disadvantages in terms of running time and accuracy. Marjanishvili [47] discussed the differences in predicted structural response for progressive collapse using both simple analysis and complex analyses. Although a complex analysis is recommended to obtain better results and realistic structural behavior, the GSA and UFC procedures allow for linear analysis because it is a cost-effective and practical approach in engineering fields. However, analytical approaches that increase accuracy of the results are always desired. Therefore, the key objective in this study is to evaluate how more accurate and detailed behavior in a concrete structure can be computed analytically.

FEMA 273/356 [5, 6] specified four analyses for buildings under earthquake forces: Linear Static Procedure (LSP), Linear Dynamic Procedure (LDP), Nonlinear Static Procedure (NSP), and Nonlinear Dynamic Procedure (NDP). Structures subjected to earthquakes are analyzed by considering lateral forces as well as gravity loads and redesigned (rehabilitated) until the acceptance criteria that were provided in FEMA 356 are satisfied. Similarly, progressive collapse analyses contain linear and nonlinear analyses. To simulate progressive collapse in a structure, notional removal of one of the critical columns is conducted depending on its exposure and location as illustrated in Chapter 2. Notional removal of a column is shown for columns on the first floor in Fig 3.1.

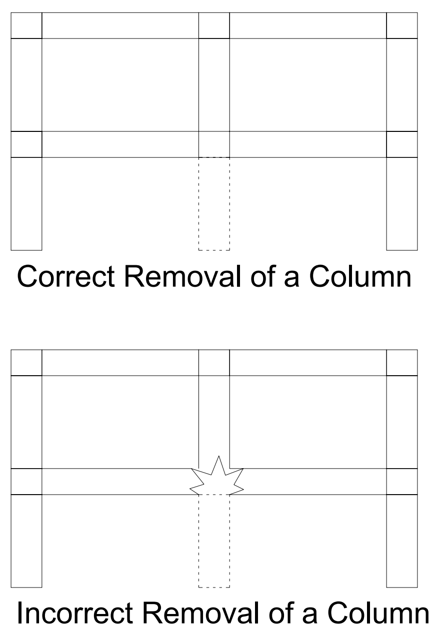


Figure 3.1 Notional removal of a column for analysis [3, 4]

When removal of a critical column is considered, two load sequences can be considered. One sequence is to initially remove a critical column and apply the gravity load (GSA). Alternatively, gravity load can be applied to the structure with all columns in place and then a critical column is removed, and the structure is analyzed (UFC).

Perform-Collapse [56], the analysis program used in this study, is based on the second load sequence because the second case better represents the manner in which loads are imposed. This analysis program can carry out linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses related to progressive collapse. Additionally, reinforced concrete structures with slabs can successfully be modeled and analyzed, and analytical results are verified with various experimental results as shown in Chapter 4.

Energy balance and dynamic factor

Perform-Collapse [56] uses an "energy balance" concept for the linear static and nonlinear static analyses. In the case a column is eliminated due to severe damage, the load resisted by the missing column loses its potential energy which must be transferred to the remaining structural system as internal energy (strain energy or inelastic energy). When potential energy is equal to the internal energy, the maximum displacement will be reached [56].

After loss of a column, gravity loads effectively increase due to dynamic effects by removal of a column. When external work by increased loads balances with internal work, the increased loads can be considered as a factor times the original gravity load. This factor is a dynamic factor to represent dynamic effects after a column is removed.

Application of the dynamic factor and energy balance is illustrated in Figs 3.2 and 3.3. When a distributed load is applied along the beams of a frame, the axial force *P*, in the center column, can be obtained from the static analysis. When the middle column (column CD) is eliminated, the remaining frame (Frame ABDFE) has to resist the distributed load as well as an inertial force due to the dynamic behavior for the entire frame system. Fig 3.3 shows the relationship between resistance of the remaining frame and displacement at D after the column CD is removed.

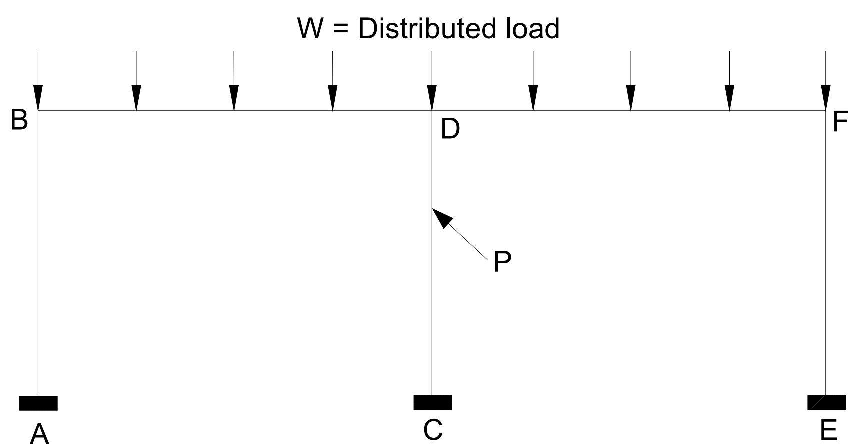


Figure 3.2 Example frame under the distributed load

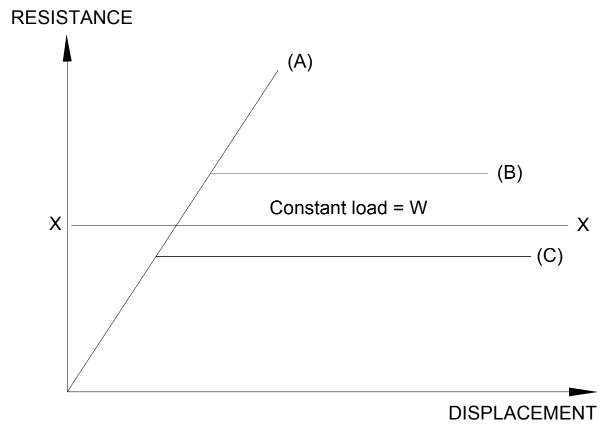


Figure 3.3 Elastic-perfectly-plastic behavior for column removal [56]

When the middle column is removed, the behavior of the frame is highly dependent on energy dissipating capacities. For case (C), the frame does not have enough capacity to reach energy balance and collapses. If the resistance is greater than the X-X dashed line, the frame can resist the constant distributed load.

For case (A), the frame behaves elastically. Therefore, it is sufficient to use a conventional linear elastic analysis in order to obtain the displacement at the removed column location. Energy balance can be reached in the elastic range. Displacement can be calculated from two load cases, which are the constant distributed load and dynamic load. Static analyses including dynamic factors and dynamic analyses produce nearly the same resulting displacement and internal forces when energy balance is attained.

Case (B) shows inelastic behavior of the frame. Depending on the energy absorption capacity of the frame, the structure may exhibit considerable ductility.

Even though the frame can resist the distributed load with gravity and dynamic load effects included, very large displacements of the adjacent members may be required to reach energy balance. However, such large deformations may exceed the acceptance criteria and the analyses should be terminated when the appropriate acceptance criteria are exceeded.

The four analysis types are discussed below in detail. In addition, modeling needed for each analysis using Ram-Perform Collapse [56] is illustrated.

3.2 LINEAR STATIC ANALYSIS

Linear static analysis generates simple, quick, and approximate results. In progressive collapse, notional removal of one of the critical columns is commonly accepted in order to simulate the progressive collapse behavior.

The GSA and UFC guidelines allow linear static analysis to obtain fast and economical results. However, it is difficult to calculate accurate behavior in a structure using linear static analysis. The GSA and UFC guidelines use a material strength increase factor up to 1.25 as discussed previously and summarized in Table 3.1. For older (pre-1970) structures where Grade 40 steel was used, a 25% increase may be excessive.

Table 3.1 Increase of the strength

Material property	Material Strength for Service Condition (ksi)	Material Strength for Progressive Collapse (ksi)
Concrete ($f_c^{'}$)	4	5
Reinforcement(f_y)	60	75

A dynamic factor of 2.0 is given for instantaneous removal of a column. The dynamic factor can initially be applied to loads as follows:

$$\text{Load} = 2\ (\text{Dead Load} + 0.25\ \text{Live Load}) \qquad (3.1)$$

Only 25% of the full live load is used to represent actual live loads in a structure. If structural behaviors are in elastic region under the given loads, calculated internal forces and displacements can be good approximations of the structural behavior because external work by the given loads are same as internal energy in the elastic region. However, if the structural response goes beyond the elastic range into the inelastic region, calculated forces from the linear static analysis exceed the capacities of the structures in inelastic regions. Additionally, displacements from the linear static analysis are smaller than results from nonlinear analysis because material nonlinearity is not considered.

Demand Capacity Ratio (*DCR*) used in FEMA 273/356 is also used for acceptance criteria for linear static analysis for progressive collapse in the GSA document.

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{3.2}$$

Where Q_{UD} is demand forces calculated for each component (moment, shear, and axial forces) and Q_{CE} is expected unfactored capacity for each component. Allowable *DCR* values in Table 3.2 are recommended in the GSA guideline. If *DCR* values exceed these criteria, that structure is at risk for progressive collapse.

Table 3.2 DCR for acceptance criteria

Structural configuration	Demand Capacity Ratio (*DCR*)
Typical structure	2.0
Atypical structure	1.5

In the Table 3.2, a “typical” structure indicates one without horizontal and vertical irregularities. An “atypical” structure includes structural irregularities in horizontal or vertical directions. A structure with re-entrant corners can be considered as an atypical structure. Structural irregularities are discussed in Chapter 6.

Elastic beams, columns, and slabs are modeled. Simple linear material properties for concrete and reinforcement are selected. Stiffness degradation can be considered by using reduction factors. FEMA 356 provides effective stiffness (degradation) values for structural components as shown in Table 3.3.

Table 3.3 Rigidity for the member forces

Component	Flexure	Shear	Axial
Beams	$0.5E_cI_g$	$0.4E_cA_w$	E_cA_g
Columns	$0.7\ E_cI_g$	$0.4E_cA_w$	E_cA_g
Slabs(two way)	$0.25\ E_cI_g$	$0.4E_cA_w$	E_cA_g

Where, E_c is the elastic modulus of concrete, I_g is moment of inertia of the gross section, A_w is area of the web section, and A_g is area of the gross section.

3.3 LINEAR DYNAMIC ANALYSIS

Linear dynamic analysis can be used to study behavior after instantaneous removal of a column. The GSA guideline specifies the following loads for such an analysis.

$$\text{Load} = \text{Dead Load} + 0.25\ \text{Live Load} \tag{3.3}$$

A dynamic factor (2.0 in the GSA guideline) is used only to simulate dynamic effects in a static analysis.

In the Perform-Collapse program [56], step-by-step integration through time using the average acceleration method (the Newmark β method) is conducted. Viscous damping is assumed to be zero in the collapse program. In

Perform-Collapse [56], the forces in the removed elements are applied to the removed location as dynamic loads, calculating the dynamic response. Therefore, structural mass can be a key factor to simulate the dynamic response when a column is missing. Perform-Collapse provides two selections for defining masses: by inputting masses on the nodes considering a tributary area of each element and automatically calculating masses for the originally distributed gravity load. The former case was selected from several analysis trials because it generated more reasonable results.

Dynamic deflections are compared in Fig 3.4 for elastic dynamic and inelastic dynamic analyses.

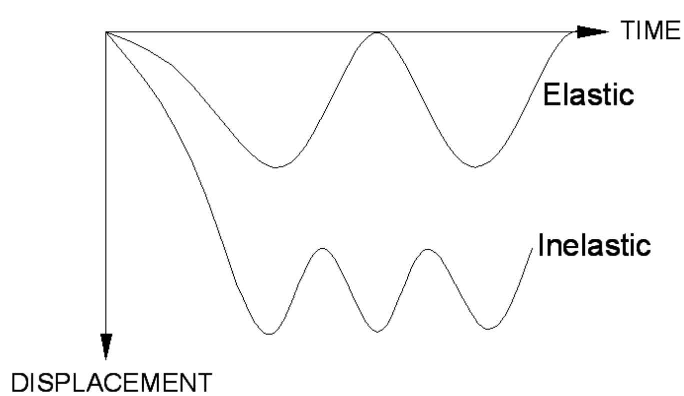


Figure 3.4 Dynamic response of the frame

Appropriate time steps should be used to obtain reliable results. Appropriate selection in the time step for nonlinear dynamic analysis is critical because too large a time step can make it difficult to reach convergence, and the results may be unreliable.

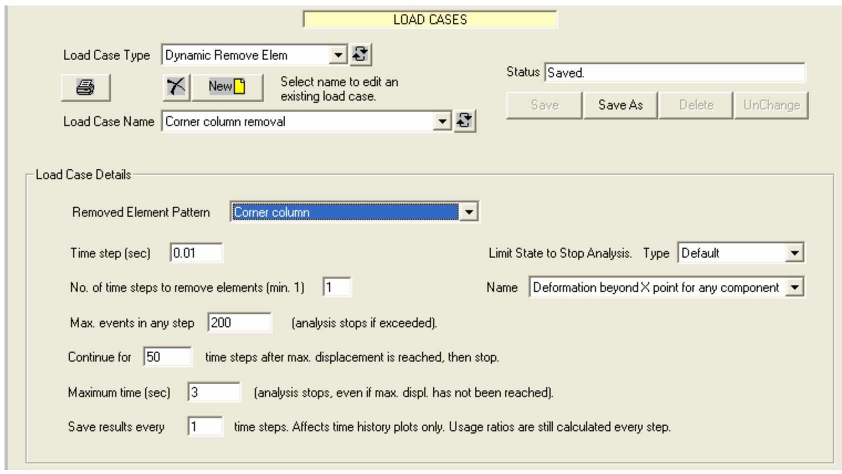


Figure 3.5 Setting of the "dynamic removed" load case

In Perform-Collapse [56], the dynamic removed load case as shown in Fig 3.5 is demonstrated as follows. The specific numbers were used following recommendations from the user's guide during analyses.

1. In Fig 3.5, a time step of 0.01 sec is selected. A user should define an appropriate time step by reducing or increasing time steps until there is no change in the results. After a column is removed and loads are applied dynamically, the maximum displacement occurs approximately at 0.5T, where T is a natural period of a structure. The manual [56] indicates 100 times steps up to maximum displacement are appropriate and the users can change the time steps until analytical results are not significantly

sensitive. Very small difference in the results for a modeled structure was found when a time step of 0.005 sec was used and this smaller time step required extremely longer computational time. An explosion against a structure typically shows duration well below 0.1 of a sec [59]. Use of 0.01 for the time step can be reasonable.

2. The number of time steps to remove element was selected as “1” that represents instantaneous removal of a column at the first time step. The user can also specify progressive removal if desired.
3. In Fig 3.5, maximum event in any step indicates “200” that represents sub-steps for each time step. The manual indicates that 200 sub steps will be sufficient to obtain convergence.
4. To study dynamic response after peak deflection is reached, 50 additional time steps after the maximum deflection is reached were used in this study, as shown Fig 3.5. If the user wants to see additional results, a large number can be specified.
5. Maximum time considered was 3 seconds. If desired, the maximum time can be increased.

Linear dynamic analysis is permitted to be used in the GSA guideline but is not permitted by the UFC. In this study, linear dynamic analysis results were compared with results from linear static analysis.

3.4 NONLINEAR STATIC ANALYSIS

In a nonlinear static analysis, vertical load is measured incrementally until the given load (with the dynamic factor) is reached or limit states (displacement, rotation, and drift limits), based on the acceptance criteria in the GSA and UFC documents, are attained.

For nonlinear static analysis using Perform-Collapse [56], vertical forces from the removed columns are applied incrementally. In a real structure, loads are applied almost instantaneously, but this is difficult to simulate in a static analysis. Material nonlinearity and geometric nonlinearity are included in the modeling components of a structure. Material nonlinearity in nonlinear static analysis is explained in Chapter 4 for verification of the program and in Chapter 5 for the structure used. Hinge properties are also defined in Chapter 5 when a frame structure is analyzed.

After a critical column is removed, dynamic factors to simulate dynamic effects should be obtained. Therefore, after removal of the critical column, a dynamic factor is imposed in the analysis by changing the applied load as follows:

$$\text{Load} = \alpha \ (1.0 \text{ Dead Load} + 0.25 \text{ Live Load}) \tag{3.4}$$

where, α is a dynamic factor.

Fig 3.6 illustrates input data of "Static removed element" of Perform-Collapse [56]: The number of load steps used in the analysis is considered to be 50 based on the manual of Perform-Collapse [56], depending on the significance of the structure and level of complexity of the structure. The maximum number of events in each step of 200 is recommended in the manual of Perform-Collapse. The dynamic factor, termed as "impact factor" in Perform-Collapse, is selected as 2.0.

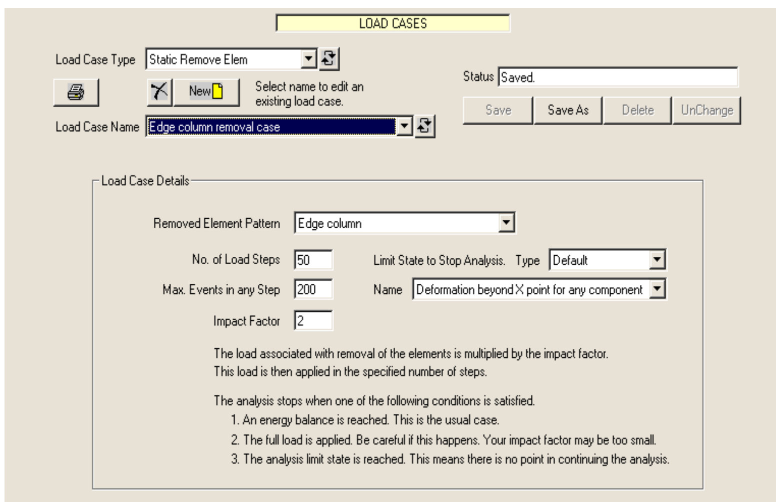


Figure 3.6 Static removed load case

3.5 NONLINEAR DYNAMIC ANALYSIS

The reliability and accuracy of results for progressive collapse analysis depend on the choice of input parameters used in the program. Input parameters were discussed in section 3.3. Therefore, small time steps require considerable time and cost. A critical column is removed dynamically and loads without any amplification factor are applied until energy balance or limit states based on the acceptance criteria are reached.

Material nonlinearity and geometric nonlinearity are checked for each time step. Changes in material properties and member stiffness are included in each time step. Viscous damping was not considered. Maximum displacement with time is the main interest in the analysis. If a complex structure is modeled with a number of segments and mesh patterns, extremely long times may be required to complete the analysis. Three story buildings modeled in this study required approximately 24hours to complete a nonlinear dynamic analysis on the Pentium 1.8Ghz computer. If finer meshes are used, much more time is required. Most input steps for the analysis procedure are identical to the linear dynamic analysis. The only difference is that nonlinearity of the structure and plastic hinges for moment and shear force should be checked at each time step to obtain more accurate results.

Using the introduced analyses, the Perform-Collapse program [56] is verified by conducting nonlinear static and dynamic analyses in Chapter 4. Linear static and dynamic analyses are conducted and compared for a robust structure in Chapter 5. Nonlinear static and dynamic analyses are conducted, and analytical results are compared for robust and deficient frame structures and deficient flat plate structure in Chapter 5 and 7.

CHAPTER 4

Verification of the analysis program

Analytical results using nonlinear static and nonlinear dynamic analyses need to be verified with results from experiments or for simple systems with well-known behavior in order to use such analyses for more complex systems.

Nonlinear static analyses for three one-way reinforced concrete slabs [32, 41, 61] and a two-way reinforced concrete slab [34] were carried out, and analytical results were compared with experimental data.

Dynamic analyses were conducted to determine response when a column of a frame structure was removed [42]. Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses for steel frame structure were conducted, and analytical results were compared with results reported in reference [42].

Material properties for concrete and steel reinforcement used in the analyses, and procedures for modeling beams, slabs, and columns are described for each case.

4.1 ONE-WAY REINFORCED CONCRETE SLAB

One-way slabs with simply supported and fully restrained boundary conditions, tested by Fenwick and Dickson [32], are compared with analytical data. A concentrated load was applied at the center of the slab. A reinforcement ratio of 0.5 percent was provided in both directions. Elevation and plan views of the slab are shown in Fig 4.1.

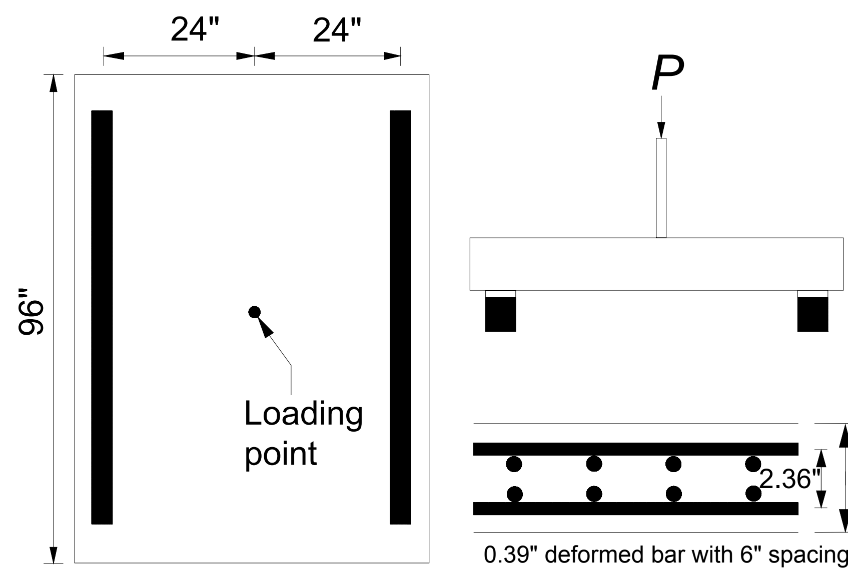


Figure 4.1 Layout of the one-way slab [32]

Material properties for the concrete and steel used when tests were conducted are given in Table 4.1. Different concrete properties used by Fenwick and Dickson are shown for the simply supported and fully fixed cases. The layered slab modeled is illustrated in Fig 4.2. The Perform-Collapse program provides up to seven default layers for concrete material and six for steel materials. Each concrete and steel layer follows a given or assumed material behavior. Tension capacity of concrete was taken from Table 4.1. The tensile strain was determined from graphs shown in Reference 45. For modeling the slab cross section, the slab was divided into five concrete layers and two steel layers for each direction shown in Fig 4.2. Load-deflection response was calculated at the center of the slab and is shown in Figs 4.3 and 4.4 for both boundary conditions.

Table 4.1 Material property of the slab [32]

Material	Properties	
	Simply supported	Fully fixed
Concrete	$f_c^{'} = 4.6\ ksi$ $E_c = 3785\ ksi$ $f_r = 0.63\ ksi$	$f_c^{'} = 3.3\ ksi$ $E_c = 3118\ ksi$ $f_r = 0.72\ ksi$
Steel	$f_y = 44\ ksi$	

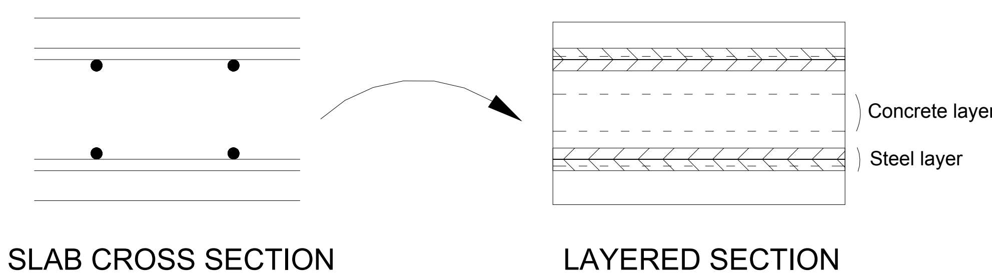


Figure 4.2 Modeling of the slab cross-section

Analytical results shown in Fig 4.3 are in agreement with the experimental results. The simply supported slab failed in punching shear when the applied load reached 34.8 kips at a deflection of 1.8 in. The fully restrained slab also failed in punching shear when the deflection reached 0.3 in at a load of 60.7 kips in Fig 4.4.

Stresses in most of the reinforcement were less than yield, and the analytical results were close to the reported experimental results. However, punching shear failure could not be simulated using the Perform-Collapse analysis program [56], which is intended to be used with structures that are controlled by flexural response.

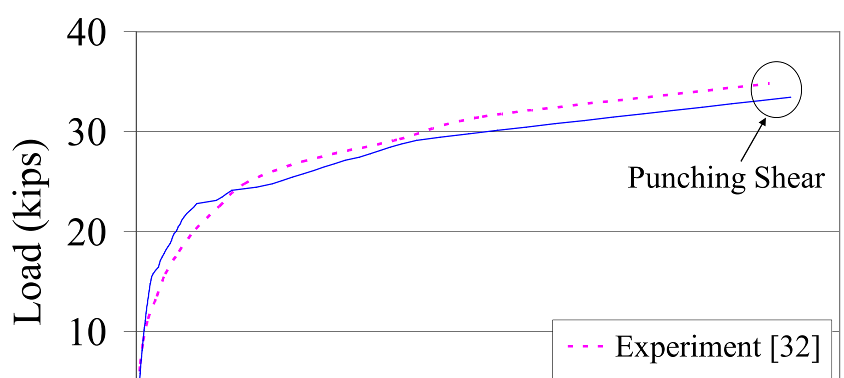


Figure 4.3 Load-deflection for the simply supported slab

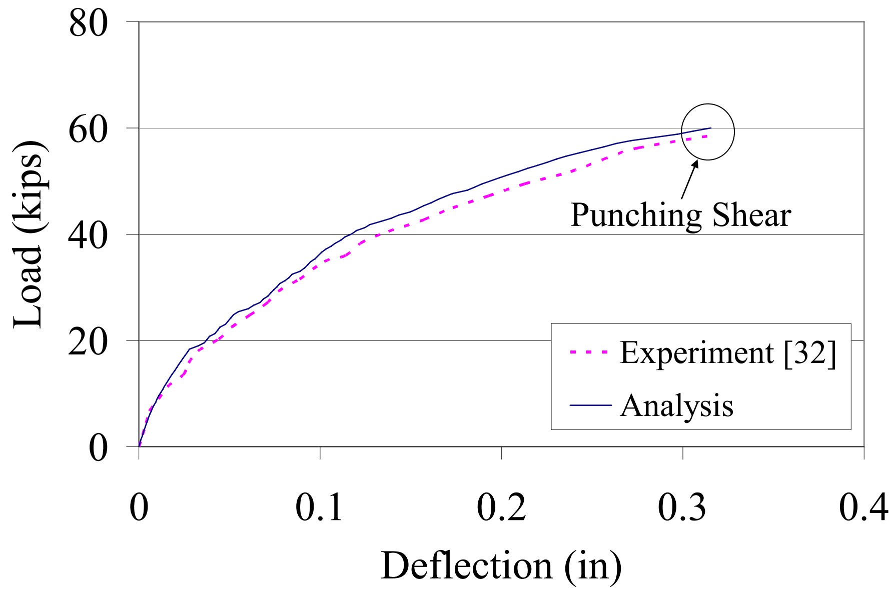


Figure 4.4 Load-deflection for the fully restrained slab

4.2 SIMPLY-SUPPORTED DUCTILE SLAB

The results of two experimental studies [41, 61] of slab structures were selected to compare with computational data calculated using Perform-Collapse [56]. Each slab exhibited ductile behavior, and deflection was recorded at the center of the panel under cyclic loading. Boundaries around the perimeter were assumed to be simply supported. A transverse load was applied at the center of the panel. Material properties for the slab are given in Table 4.2. The shape and details of the slab are shown in Fig 4.5. Slab types 1 and 2 had different steel configurations.

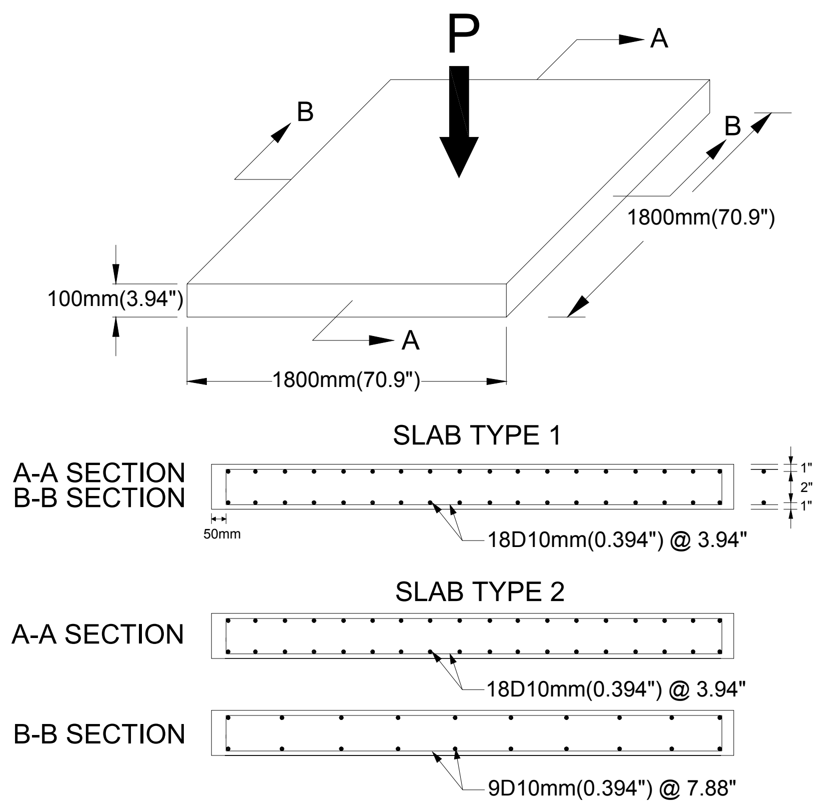


Figure 4.5 Shape and details of the slab

Load-deflection curves of the slab at the center location are shown in Fig 4.6. The load carrying capacity of slab type 2 was lower than that of slab type 1 because spacing of the bottom reinforcement was greater for slab type 2.

Table 4.2 Material properties of the slab

Material	Properties
Concrete	$f_c = 5.4\ ksi$
Steel	$f_y = 55\ ksi$

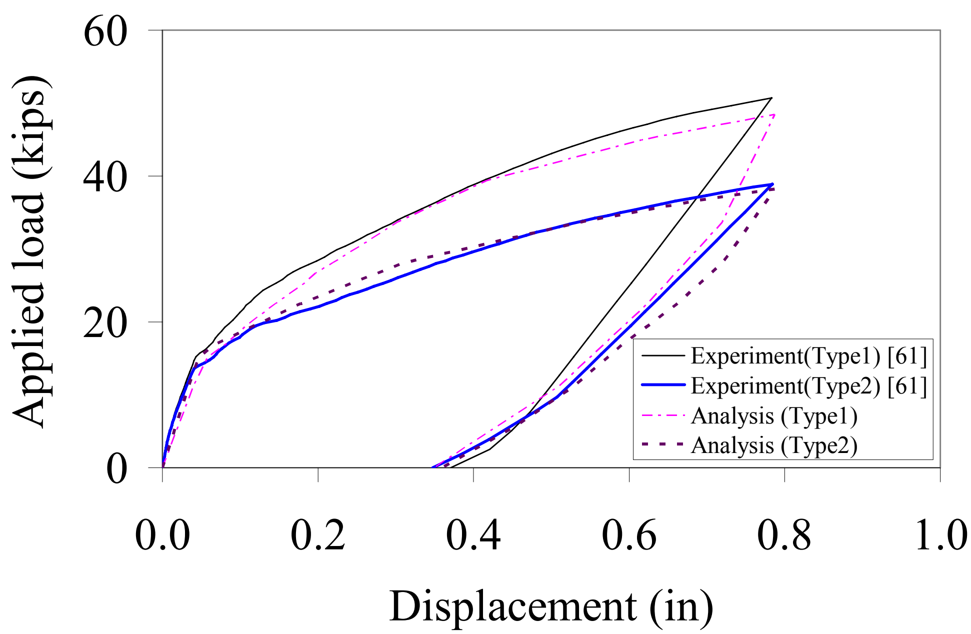


Figure 4.6 Load-deflection of the slab at the center location

The slabs did not fail in shear, and good agreement between experiment and analyses were observed, even under the cyclic loading. Perform-Collapse [56] generated reasonably accurate data. However, the structures modeled had simple shapes and well defined boundary conditions. Comparisons with more complex structures are needed. Therefore, a nine panel slab with spandrel beams and interior beams was modeled and compared.

4.3 TWO-WAY CONCRETE PANEL SLAB [34]

The nine-panel reinforced concrete slab was a quarter-scale model of the prototype slab [34]. The model structure consisted of spandrel beams with 3 in by 4.25 in and interior beams with 3 in by 5 in. The slab was 1.5 in thick. Material properties for the structural members are summarized in Table 4.3. Plan and elevation views are shown in Fig 4.7 and Fig 4.8. Details of the reinforcement in the slab are shown in Fig 4.9.

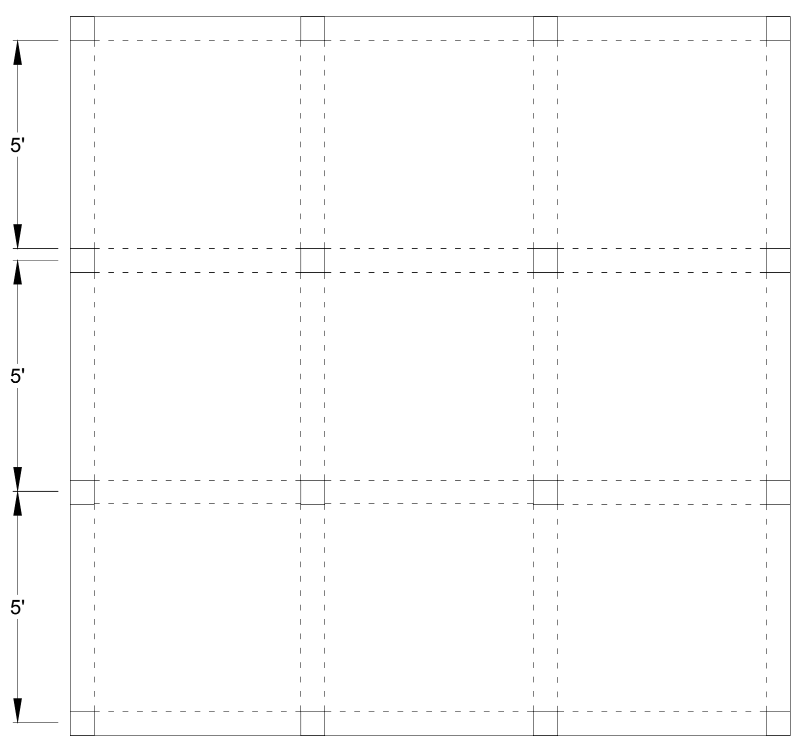
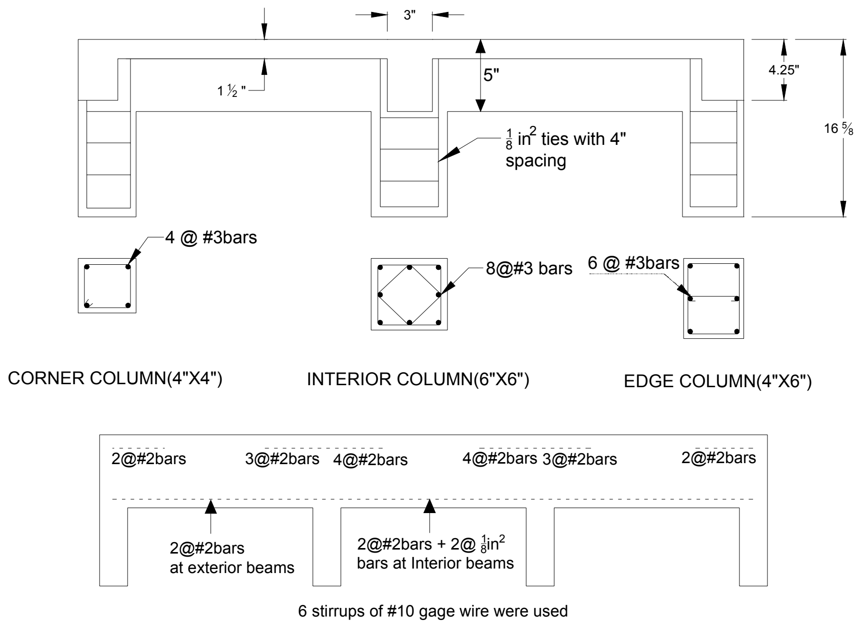


Figure 4.7 Plan view of the nine panel slab [34]



Beam reinforcement

Figure 4.8 Elevation view of the nine panel slab and beam [34]

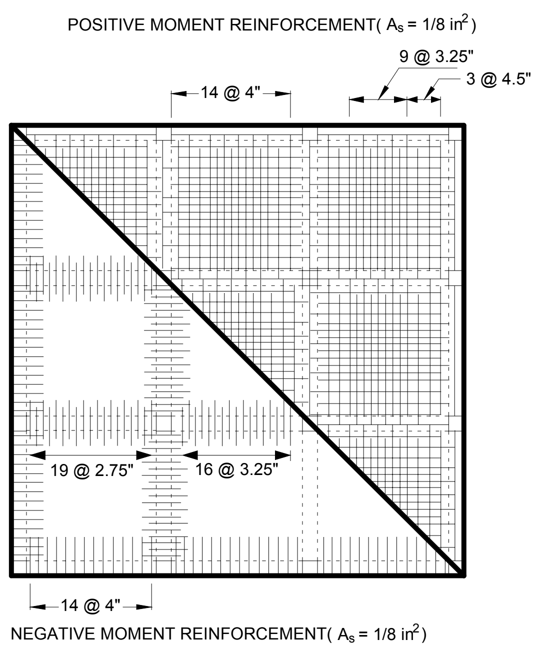
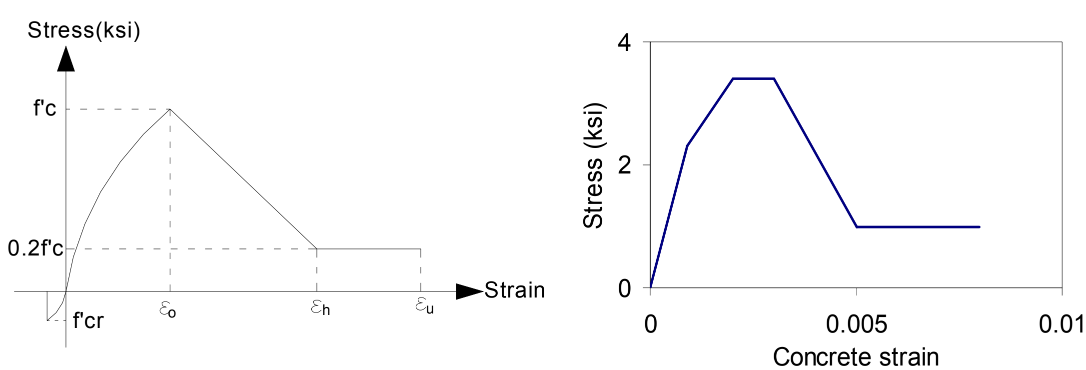


Figure 4.9 Reinforcement distribution of the slab [34]

Table 4.3 Material properties for the nine panel slab [34]

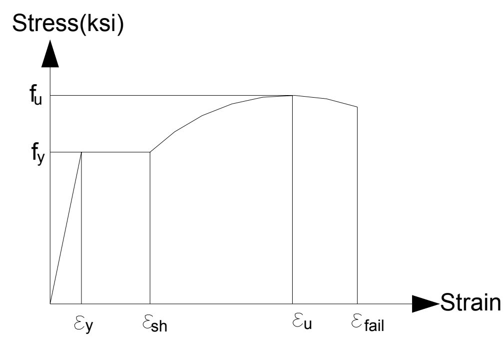
	Beams	Columns	Slabs
Concrete	$f_c = 3420\ psi \pm 230\ psi$ (4x8 cylinders) $f_c = 2830\ psi \pm 310\ psi$ (2x4 cylinders) $f_r = 590\ psi \pm 58\ psi$		
Longitudinal Steel	$No.2\ bars$ $f_y(avg) = 50\ ksi$	$No.3\ bars$ $f_y(avg) = 55\ ksi$	$1/8\ in^2\ bars$ $f_y(avg) = 42\ ksi$ $f_u(avg) = 59\ ksi$
Transverse steel	# 10 gage wire $f_y(avg) = 40\ ksi$	$1/8\ in^2\ bars$	-

Based on the given material properties, the stress-strain curves for concrete and steel were established as shown in Fig 4.10 and 4.11. In Figs 4.10(a) and 4.11(a), stress-strain relationships often used for each material are plotted [54]. In Figs 4.10(b) and 4.11(b), the stress-strain curves used in the program are shown. Linear curve fitting was needed because the program does not permit curved segments in the stress-strain curves.

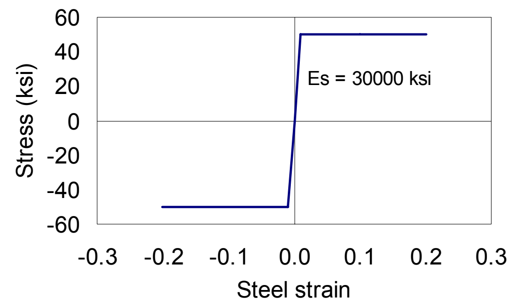


(a) (b)

Figure 4.10 Stress-strain relationship for concrete [54]



(a)



(b)

Figure 4.11 Stress-strain relationship for steel [54]

Most structural components in an analysis program are modeled using a center line for each component. Modeling using center line dimensions may be valid for a monolithic plane frame structure. However, there will be discrepancies in the results when a beam-column frame with slabs is modeled because center line modeling can not accurately represent the geometric differences between beams and slabs. A concrete beam with a composite floor slab is shown in Fig 4.12.

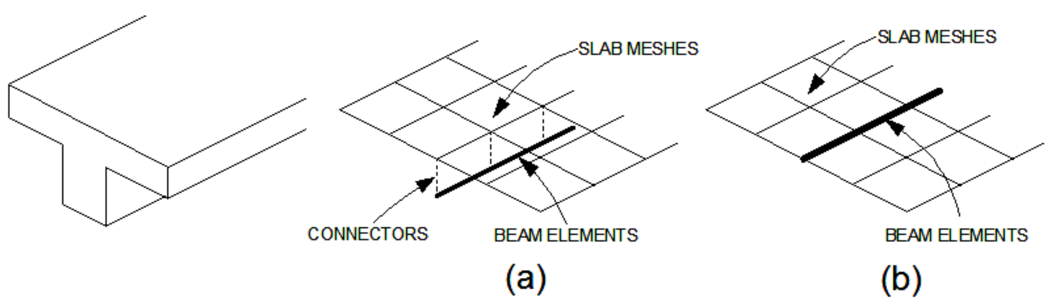


Figure 4.12 Modeling of a floor [56]

There are two ways to model this type of structural component: Case (a) and Case (b) in Fig 4.12.

The first case [Case (a)] considers nodes for slabs at mid-thickness and beams at the centroid. This model requires additional connectors to link beams and slabs as shown in Fig 4.12 (a). These connectors can be made as an additional short and stiff beam or column. If shear slip between beams and slabs is expected, shear hinges can be considered. The disadvantage of this model is that it requires very large number of nodes. It is also difficult to define appropriate properties for connectors. Connectors that are too stiff generate numerical sensitivity problems [56].

The second case [Case (b)] provides a simpler modeling assuming rigid connection between beams and slabs but without shear connectors. This model uses fiber cross section for beams. The reference axis for the beam cross section is defined to be at the mid-thickness of the slab. Therefore, concrete and steel properties and locations should be placed relative to the reference axis. This model makes it simple and convenient to establish composite section of beams and floor slabs and is verified by analyzing a two-way slab (nine panel slab test). Implementation of this model required communication with Dr. Graham Powell[1] who developed the program for RAM International.

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[1] Private communication with Dr. Graham Powell, RAM International.

Some modifications to the model were suggested by Dr. Powell. Using Perform-Collapse, modeling of the nine-panel slab was established by considering a quarter of the slab using lines of symmetry as shown in Fig 4.13.

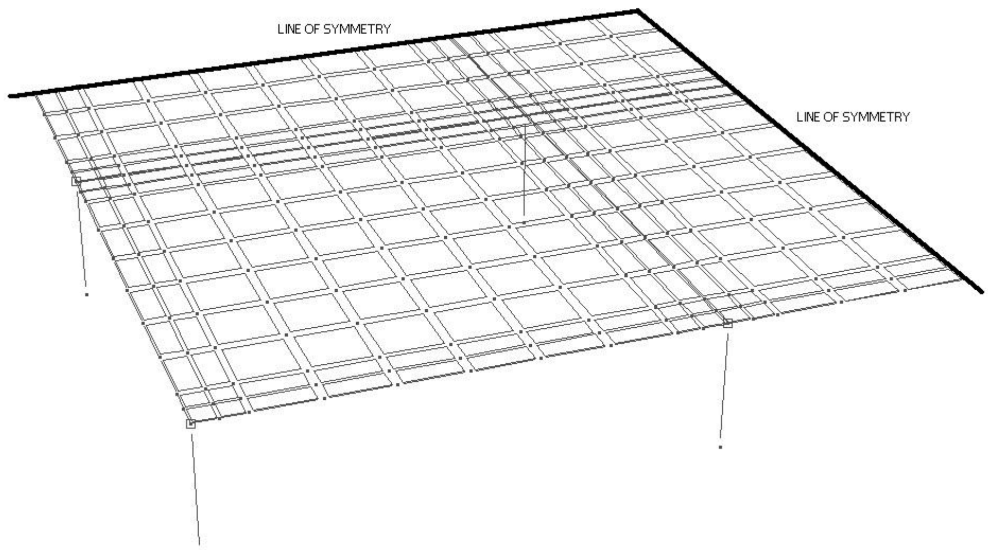


Figure 4.13 Meshes of the floor slab

Each slab mesh was constructed using top and bottom reinforcement configurations. Finer meshes were used at the corner, edge, and interior column locations to improve accuracy and to obtain detailed behavior of the structure. Hinged supports were used at the bottom of each column and represent the supports in the original structure. Fixed ends without restraint in the vertical direction represented the condition at lines of symmetry at the center of the beams and slabs as shown in Fig 4.13. Due to the possibility of torsional damage in beams, reduced torsional rigidity was initially considered. Load deflection at the center of a corner panel is presented in Fig 4.14.

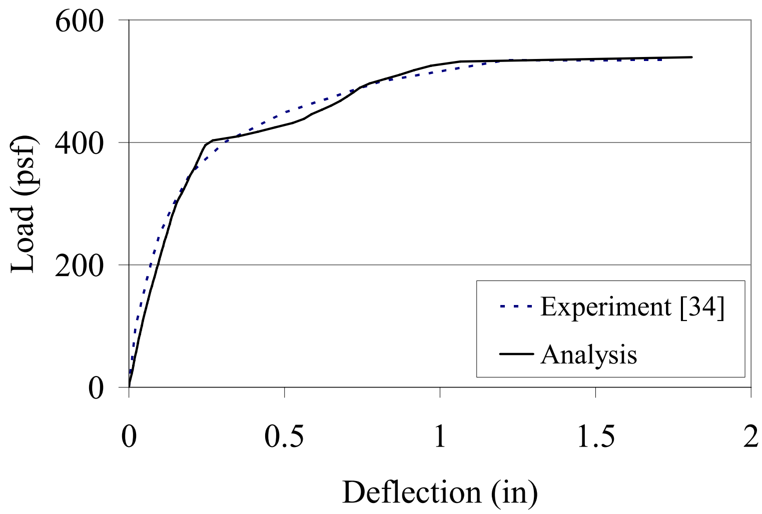


Figure 4.14 Load-deflection of the panel slab structure

When the ultimate load on the test structure was reached, deflection at the center of the corner panel was 1.72 in. Analytical results showed very good agreement with the experimental results. Some difference in deflections can be caused by various concrete strengths when the panel structure was established as indicated in Table 4.3. Concrete stress-strain relationships used in the analysis program were based on average compressive strength. Deflections at the middle of the spandrel beams and the center of the slabs are plotted in Fig 4.15. Upper values were measured from the experiment, and numbers in the parentheses were determined from the analysis in Fig 4.15. Steel properties for beam, slab, and column components were also based on average yield and tensile strengths.

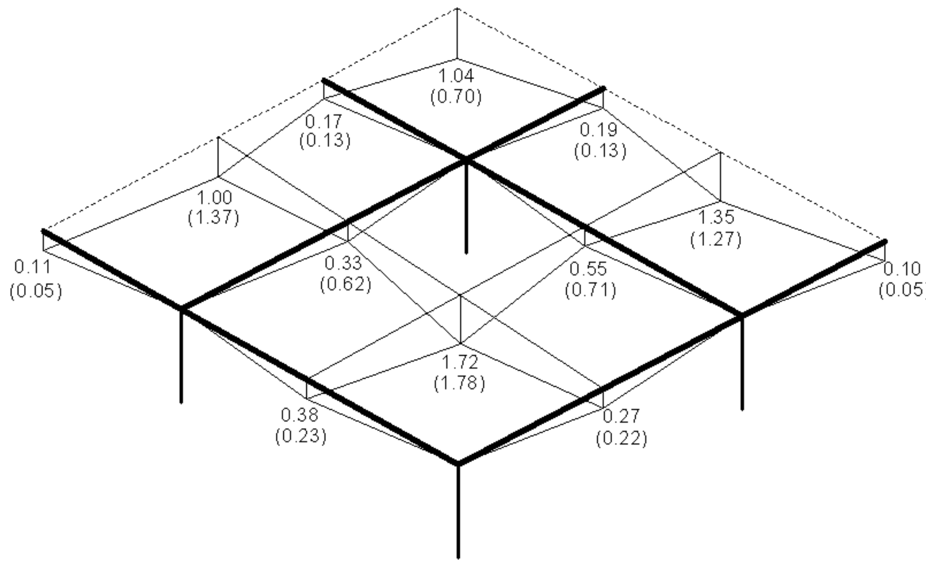


Figure 4.15 Comparison of deflection at the observed locations

The detailed behavior of the floor slab at each load level is compared in Table 4.4 and 4.5. The test results at identical or similar load levels are compared with the analytical values. First yield in the reinforcement at negative moment regions occurred at 288 psf in the experiment and 356 in the analysis. This difference may be attributed to variations of the concrete and steel, mesh sizes of the modeling, and difference between real locations of the reinforcement and the steel layers in the program. Based on comparison between analytical results and test results, Perform-Collapse [56] generated deformations that were close to experimental results. The Perform-Collapse program [56] is used for nonlinear static analysis of more complex structures in the following chapters.

Table 4.4 Experimental response of the nine panel slab structure

Load (psf)	Response of the floor slab (Experiment)
150	Maximum steel stress in the slab 2.5 ksi (Positive moment region) 3.9 ksi (Negative moment region)
213	A few cracks observed on top of slab, No cracks on the lower surface
288	First yielding at the negative moment reinforcement
311	First crack on the lower surface
353	Positive reinforcement reached yielding point
533	Maximum load (experiment stopped)

Table 4.5 Detailed observation of the structural behavior from the analysis

Load (psf)	Response (Analysis)
150	Maximum stress for positive reinforcement for slab=2.6 ksi Maximum negative steel for the slab = 3.7 ksi
194	First crack at the top slab
356	First yield in the negative moment steel around the center panel
407	First yield in the negative moment steel between the edge panel and corner panel, first yield in positive moment steels at the corner panel
432	Spread of yield in the entire negative moment region
482	First yield at the positive slab at the each center location adjacent to the corner panel
496	First yield at the negative moment steel at the spandrel beam
504	First yield at the positive moment steel at the interior and exterior beam
525	Yield of positive reinforcement in the center panel
537	Spread of hinges to all locations (program stopped)

4.4 DYNAMIC ANALYSIS FOR A STEEL FRAME

In order to simulate progressive collapse in a structure, Perform-Collapse [56] must also be verified. However, dynamic response of concrete structures resulting from removal of a critical vertical element was not found in the literature. To check Perform-Collapse [56] with other linear dynamic and nonlinear dynamic analytical studies, a frame used by Kaewkulchai and Williamson [42] in their analysis of progressive collapse was studied. The two dimensional steel frame as shown in Fig 4.16 was modeled to investigate accuracy of Perform-Collapse [56]. Four analyses (linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses) were executed using Perform-Collapse [56] to compare results with those reported in the reference 42. The analysis procedures were discussed in detail in Chapter 3. The frame configuration is shown in Fig 4.16, and structural properties are summarized in Table 4.6.

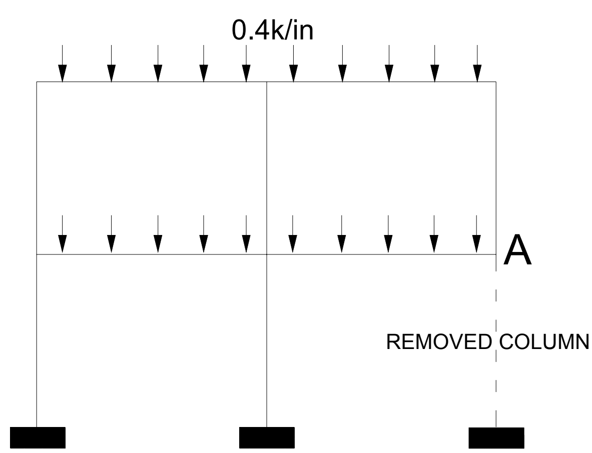


Figure 4.16 A two story - two bay steel frame configurations

Table 4.6 Beam-column properties

Properties	Beam	Column
Length	240 in	144 in
Area	12 in^2	20 in^2
Moment of Inertia	2000 in^4	1200 in^4
Plastic capacity(M_p)	6500 k-in	6500 k-in

Fig 4.16 shows the frame with a uniform load of 0.4 k/in. The columns were fixed at the foundation. Localized failures such as local flange and web buckling, warping, and lateral buckling were excluded. The current version of Perform-Collapse [56] does not provide these local failures. In order to simulate damage in a vertical element due to an abnormal loading situation, the right column at the first floor was removed as shown in Fig 4.16. In the dynamic analysis, a time step of 0.01 sec was used. Lumped beam masses for the dynamic analysis were located at each end of a beam. Damping was not considered in the dynamic analyses because most energy was dissipated by plastic hinging in the structural components. Deflections at A (Fig 4.16) from the static and dynamic analyses after the column was removed are shown in Figs 4.17 to 4.19.

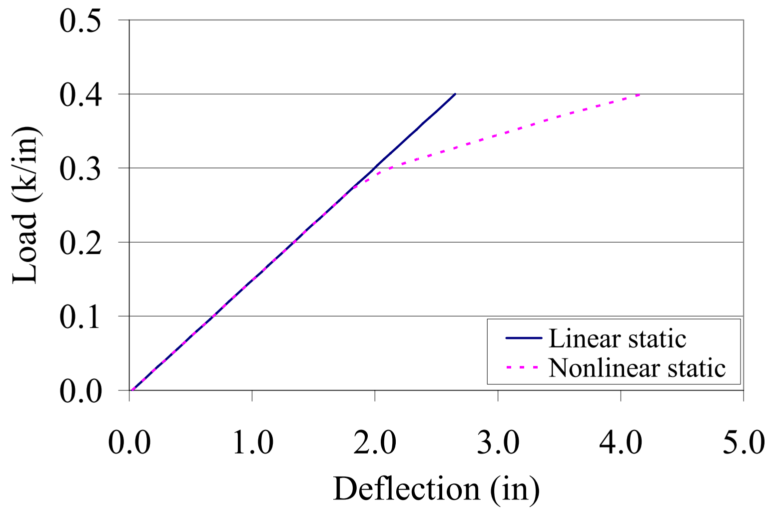


Figure 4.17 Load-deflection from the static analysis

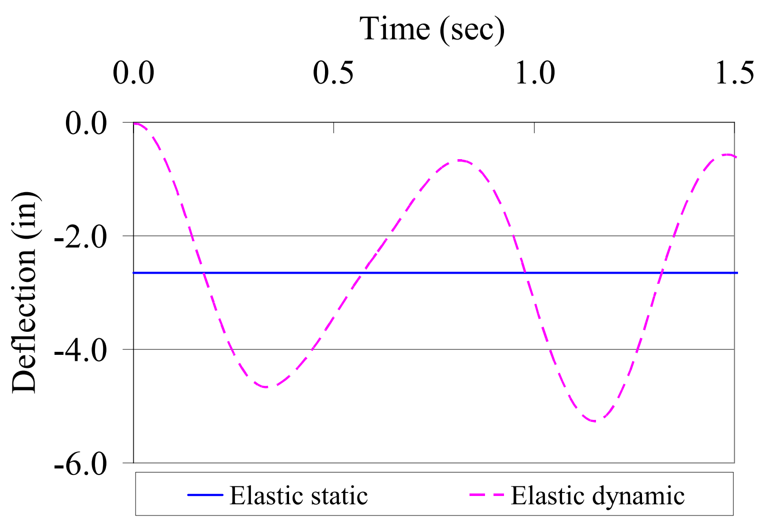


Figure 4.18 Load-deflection from the dynamic analysis

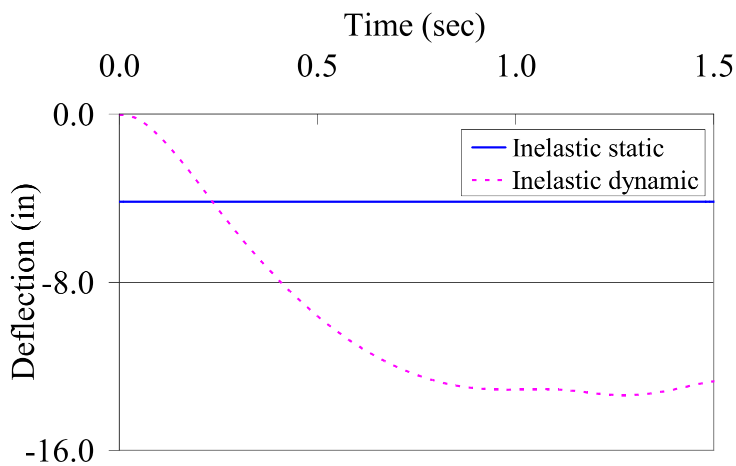


Figure 4.19 Load-deflection for inelastic dynamic analysis

Table 4.7 Maximum deflection at the target location for each analysis

Maximum deflection (in)	Elastic static analysis	Elastic dynamic analysis	Inelastic static analysis	Inelastic dynamic analysis	Inelastic static analysis with dynamic factor
Perform Collapse	2.65	5.26	4.18	13.95	15.63
Reference [42]	2.65	5.27	4.71	14.70	14.70 from inelastic dynamic analysis

Results from the Perform-Collapse program [56] showed good agreement with results reported in Reference 42. Perform-Collapse also allowed the structure to form hinges so that a mechanism formed and maximum deflection was reached. In the inelastic static analysis, the dynamic effect can be simulated using a dynamic factor. After a column was removed, external work balanced with internal work when loads representing dynamic effects reached the applied load plus additional loads (0.14 times of the applied loads). A relatively small dynamic factor (1.14) increased the load to 0.456 k/in and resulted in reaching maximum deflection (15.63 in) as shown in Fig 4.20. The initial gravity load (0.4 k/in) was multiplied by the dynamic factor (1.14) for the analysis shown Fig 4.20. This dynamic factor was much smaller than the factor that is suggested in the GSA document (2.0). The difference between the dynamic factor from the analytical results and that from the GSA guideline reflects the design perspective. Although nonlinear analyses generate smaller dynamic factors (less than 2.0), the intent of the GSA guidelines is to provide a lower bound on the factor for structural design against abnormal loads so that the design will be conservative.

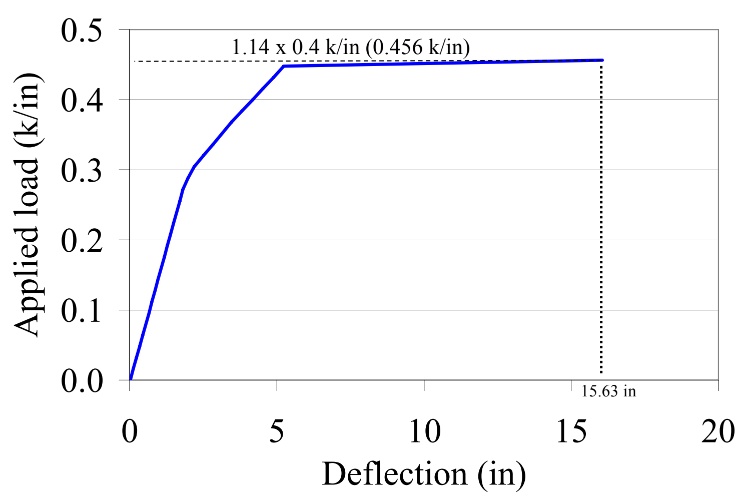


Figure 4.20 Nonlinear static analysis until energy balance reached

Hinge locations were also compared with those reported in Reference 42. From the results, inelastic static and inelastic dynamic analyses generated the same hinge formation as provided in Reference 42. Hinge formations are illustrated in Fig 4.21 and summarized in Table 4.8.

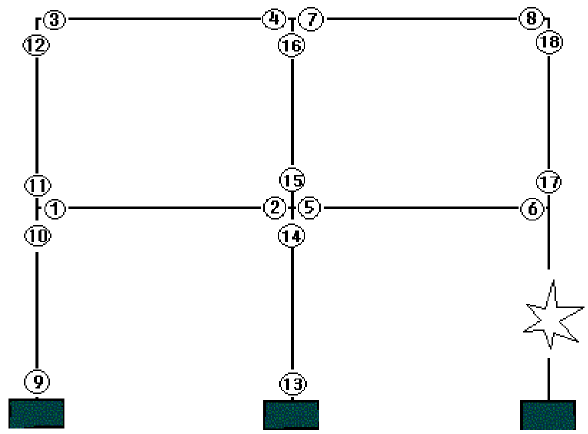


Figure 4.21 Sequence of hinge formation in the frame

Table 4.8 Hinge locations in Reference 42

Analysis	Hinge locations
Inelastic static	2,4,5,7
Inelastic dynamic	2,3,4,5,6,7,8,11,12,14,17,18

Table 4.9 Hinge locations for inelastic static analysis with the dynamic factor

Load	Dynamic factor	Hinge locations
0.4 k/in	1.14	2,4,5,6,7,8,18

Hinges at beams and columns formed at many locations in inelastic dynamic analysis, compared with inelastic static analysis from Table 4.8. This phenomenon was termed the dynamic spreading effect [42].

A smaller number of hinges formed when the inelastic static analysis with the dynamic factor was conducted as shown in Table 4.9. Although the dynamic factor was included in the inelastic static analysis, inelastic static analysis did not demonstrate dynamic spreading effect of the structure because the inelastic static analysis involves a static load that increases deflections but does not represent dynamic effects at the far ends of adjacent beams. Therefore, both inelastic static analysis with a dynamic factor and inelastic dynamic analysis need to be investigated. Inelastic static analysis represents a simpler approach to progressive collapse analysis using a dynamic factor but inelastic dynamic analysis results in a more reliable and accurate representation of structural behavior.

The Perform-Collapse program has been verified by comparing with test results for nonlinear static analysis and analytical results for nonlinear dynamic analysis. Progressive collapse analysis will be conducted using Perform-Collapse for a robust structure in Chapter 5 and a deficient structure in Chapter 7.

CHAPTER 5

Robust Structure

Beam-column frame structures with spandrel beams were modeled and analyzed. For each structure, structural conditions were classified as robust or deficient. A robust structure is one built following current code requirements and satisfying the detailing provisions for high seismic risk. A deficient structure can be defined as an existing concrete structure constructed before the more stringent provisions for seismic resistance and structural integrity in current code requirements were developed. The following deficiencies are common in existing concrete structures designed before 1976 for gravity load only: short embedment length of bottom beam reinforcement (positive moment) at column supports, insufficient transverse reinforcement for shear when load patterns significantly change, and compression splices in columns where tension may occur if gravity load carrying elements are severely damaged. These deficiencies may prevent the development of alternate load paths when elements are damaged or destroyed and may lead to progressive collapse. A robust structure with a lateral force resisting system does not have these deficiencies. Detailed behavior of the elements in a robust structure was investigated following removal of critical load-carrying elements. Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses were conducted for each structure. Factors to simulate dynamic effects suggested by the GSA and UFC guidelines were also investigated.

5.1 BEAM-COLUMN FRAME STRUCTURE

A three-story concrete structure used in the PCA design handbook [31] is shown in Figs 5.1 and 5.2 and was taken as the prototype structure. The three-story reinforced concrete frame had 18 in by 18 in beams and columns and 7-in thick slabs. Although it is not often for a designer to use a square beam, beams were modeled following the design dimension given in the PCA design handbook. The concrete building was originally designed to resist gravity, wind, and seismic loads. The structure was designed using the seismic detailing design provisions of ACI 318-95. Reinforcement details for slabs at the column strip and middle strip locations and for beams at exterior and interior locations are provided in Figs 5.3 and 5.4.

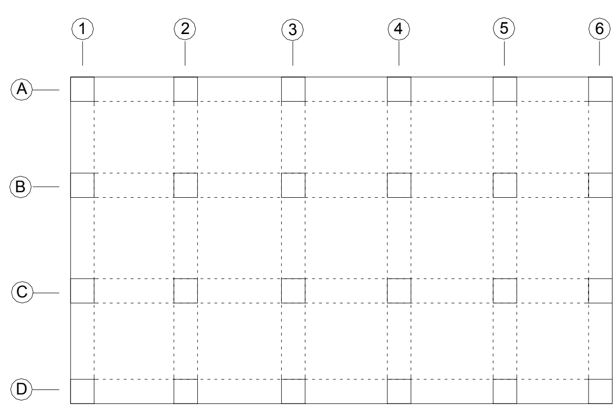


Figure 5.1 Plan view of the concrete frame [31]

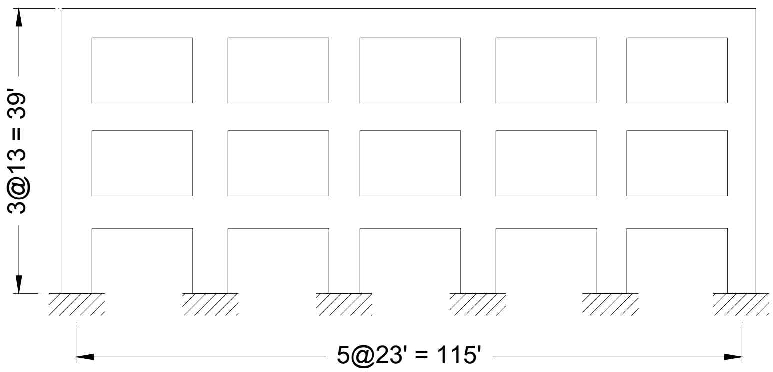
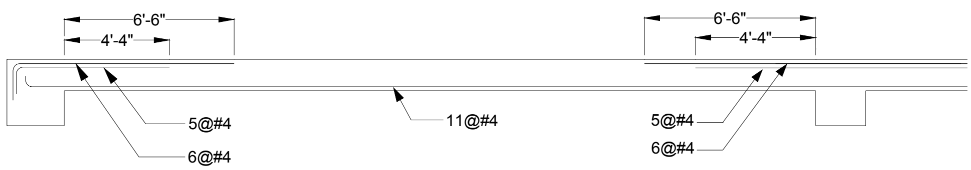
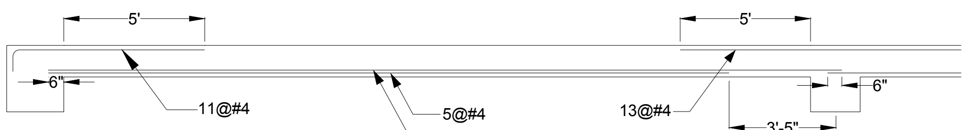


Figure 5.2 Elevation view of the concrete frame [31]



Column strip



Middle strip

Figure 5.3 Reinforcement details in a slab at column strip and middle strip [31]

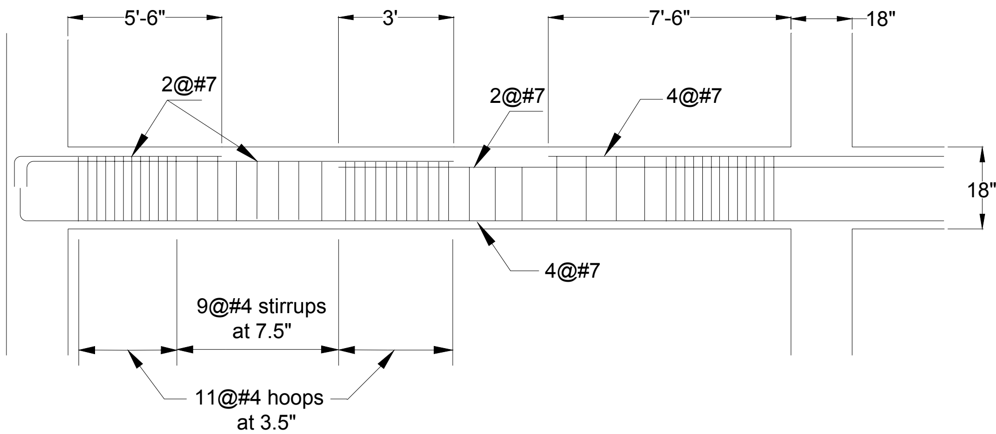


Figure 5.4 Reinforcement for interior and exterior beams [31]

Exterior columns used 8-#6 bars and interior columns 8-#9 bars because the sum of column flexural strength at interior joints should be greater than or equal to 6/5 times the sum of the flexural strengths of the beams based on ACI seismic provision. Column cross sections at exterior and interior frames are shown in Fig 5.5.

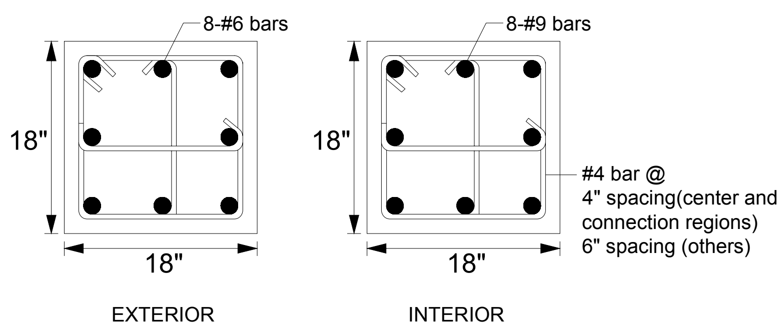


Figure 5.5 Column cross sections

Material properties used for design are provided in Table 5.1. The original design loads including dead, live, seismic, and wind loads are summarized in Table 5.2.

Design of the robust structure was based on factored load combinations with lateral loads. Load combinations used for the design of the structure were based on both ACI 318-95 and SBC 1997 (Standard Building Code) [8] and summarized at Table 5.3.

Table 5.1 Material properties [31]

Concrete	$f_c^{'} = 4000\ psi$ $w_c = 150\ pcf$ $E_c = w_c^{1.5}\ 33\sqrt{f_c^{'}} = 3830\ ksi$
Reinforcement	$f_y = 60000\ psi$

Table 5.2 Load on the floor [31]

Dead load	87.5 psf + 30 psf (Superimposed) = 117.5 psf (Floor) 87.5 psf + 10 psf (Super imposed) = 97.5 psf (Roof)
Live load	60 psf (Floor), 30 psf (Roof)
Seismic Design Data	Seismic hazard exposure group : I $A_v = A_a = 0.25$ Site Coefficient for soil type $S_2 = 1.2$
Wind design Data	Basic wind speed = 70 mph Exposure category *C* Wind load importance factor $I = 1$

A_v : Effective peak velocity - related acceleration
A_a : Effective peak acceleration

Table 5.3 Load combinations used in design

Code	Equation
ACI 318-95	$1.4\,D + 1.7\,L$ $0.75\,(1.4\,D + 1.7\,L \pm 1.7\,W)$ $0.9\,D \pm 1.3\,W$
SBC 1997	$(1.1 + 0.5\,A_v)\,D + L \pm Q_E$ $(0.9 - 0.5\,A_v)\,D \pm Q_E$

D = Dead loads
L = Live loads
W = Wind loads
Q_E = Horizontal seismic loads
A_v : Effective peak velocity - related acceleration

The design base shear for the seismic load and wind load based on information given in Table 5.2 are provided in Table 5.4.

Table 5.4 Design base shear for seismic and wind loads

Load	Design Base Shear
Seismic load	258 kips
Wind load	73.1 kips

5.2 ROBUST STRUCTURE

A robust structure is defined as a concrete structure built following current ACI code provisions considering structural integrity and seismic detailing. The robust structure shown in Fig 5.6 had continuous bottom reinforcement along beams and in the area of connections. Good transverse reinforcement details for shear were also provided. In addition, adequate compression splice lengths in columns were provided to resist potential tension forces in columns resulting from load redistribution if a column is removed. Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses were conducted for cases with columns removed as shown in Fig 5.7.

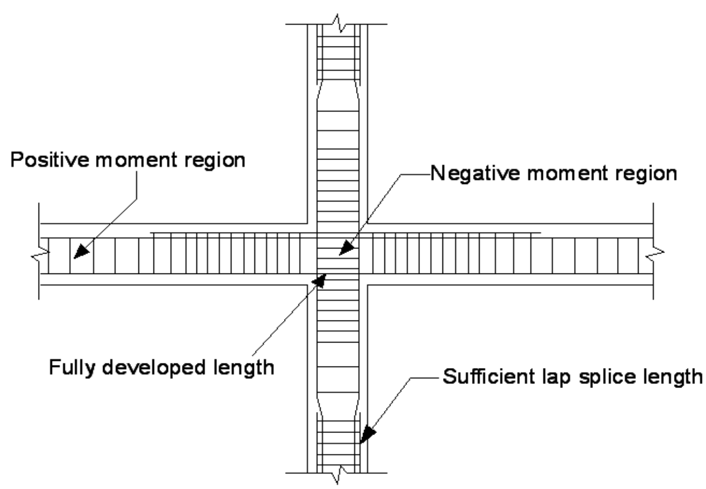


Figure 5.6 Robust structure built in the current regulation

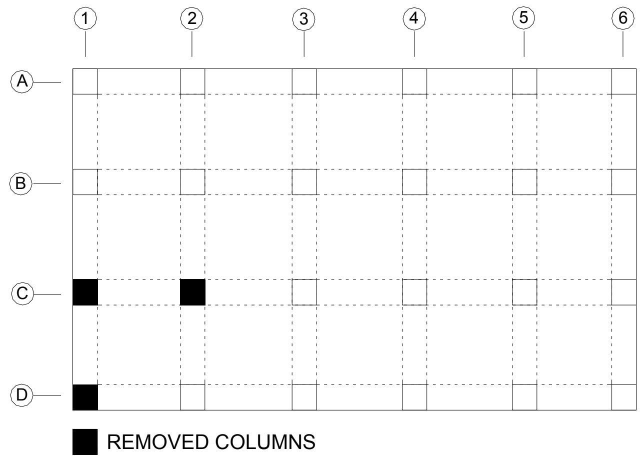


Figure 5.7 Removed columns in the frame

The design load for progressive collapse analysis is shown in Table 5.5. Typical design load multiplied by load factors for dead and live loads is compared.

Table 5.5 Design load

Design load for Progressive Collapse	Dead load + 0.25 Live load 144 psf (Floor), 115 psf (Roof)
Typical design load (ASCE, ACI)	1.2 Dead load + 1.6 Live load 252 psf (Floor), 177 psf (Roof)

5.2.1 Load sequence

When a critical column is removed, the sequence of loading used in this study is different from the GSA and UFC guidelines. The GSA guideline provides a factor to represent dynamic effects and required demand capacity ratios (DCR), a ratio of the demand to the expected capacity in a member needed for critical load bearing elements. A factor of 2.0 is used in a static analysis to reflect the dynamic effects of removal of a column.

In order to analyze a concrete structure based on the GSA guideline, a column on the first floor is removed (the UFC guideline considers removal of a column at other floors as well as at the first floor). Both guidelines consider notional removal of critical columns. The steps in conducting a linear analysis based on the GSA guideline are as follows:

- Initially remove a critical column.
- Apply the gravity load given in Table 5.6 (using a dynamic factor of 2.0 for static analysis).

Table 5.6 Load based on the GSA

Analysis	Load
Static	2 (DL + 0.25 LL)
Dynamic	DL + 0.25 LL

- Determine if DCR is exceeded.
- From the GSA guideline, if DCR is more than 2.0, a structure has a possibility of progressive collapse.

For nonlinear static analyses using Perform-Collapse, dynamic effects were evaluated by determining dynamic factors when each column is removed. A different load sequence is recommended to simulate actual loading situation [56].

Load sequence used in this study is as follows:

- Apply a design load (ex. 144 psf for floors).
- Remove one of the critical columns.
- Increase gravity load multiplying by a dynamic factor (α) until external work reaches internal work.

Therefore, after removal of a column, the load equation can be:

$$\text{Load} = \alpha \text{ (Design Load)} \tag{5.1}$$

where, design load = (Dead Load + 0.25 Live Load)

- Determine possibility of progressive collapse considering limit states.

5.2.2 Linear analysis in the GSA and UFC guidelines

The GSA and UFC guidelines recommend that nonlinear dynamic analyses generate the most accurate analytical results. Nonlinear dynamic analysis requires complexity of the analysis and considerable computational time. Therefore, the GSA and UFC guidelines allow for the use of linear static analysis with a dynamic factor based on nonlinear dynamic analysis.

Linear static and dynamic analyses were conducted using Perform-Collapse [56], and analytical results for the linear static and dynamic analyses are summarized in Table 5.7 and 5.8. Deflections in Table 5.7 and 5.8 were observed at the point above which a column was removed. Loads equal to and larger than the design load (144 psf) for progressive collapse analysis were investigated. A larger load than 144 psf takes into account the conventional design load when typical load factors (1.2 for dead load and 1.6 for live load) are used, as indicated in Table 5.5.

For the effects of cracking, half of the elastic modulus of the beams was used. Load-deflection response from the linear static and dynamic analyses is shown in Figs 5.8 and 5.9. The deflections were captured at the point right above the removed column.

Table 5.7 Deflection and dynamic factor for 144 psf

Removed location	Linear Analysis	Load	Deflection (in)
Corner Column (D1)	Static*	***2.0*** x 144 psf	*1.22*
	Dynamic	144 psf	*1.18*
Edge Column (C1)	Static*	***2.0*** x 144 psf	*1.37*
	Dynamic	144 psf	*1.36*
Interior Column (C2)	Static*	***2.0*** x 144 psf	*1.68*
	Dynamic	144 psf	*1.67*

* With dynamic factor of 2.0

Table 5.8 Deflection and dynamic factor for 252 psf

Removed location	Linear Analysis	Load	Deflection (in)
Corner Column (D1)	Static*	***2.0*** x 252 psf	*2.07*
	Dynamic	252 psf	*2.00*
Edge Column (C1)	Static*	***2.0*** x 252 psf	*2.31*
	Dynamic	252 psf	*2.30*
Interior Column (C2)	Static*	***2.0*** x 252 psf	*2.83*
	Dynamic	252 psf	*2.82*

* With dynamic factor of 2.0

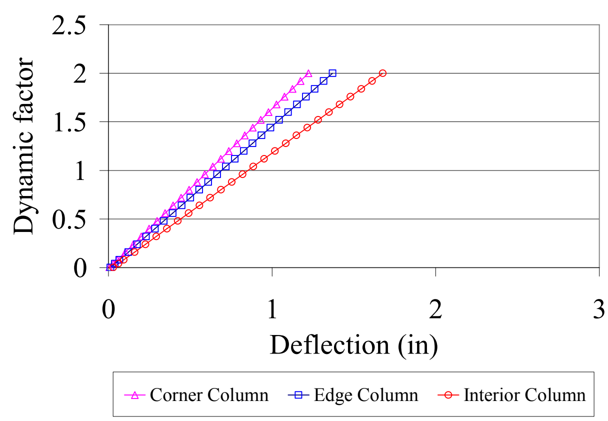


Figure 5.8 Linear static analysis for 144 psf

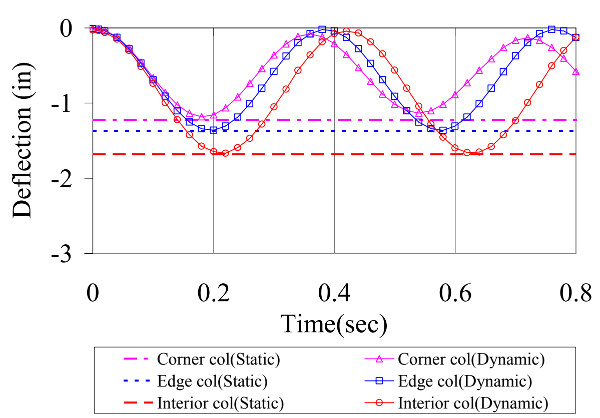
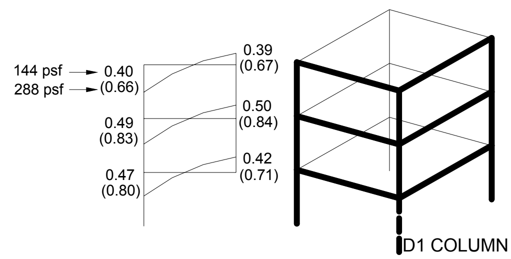


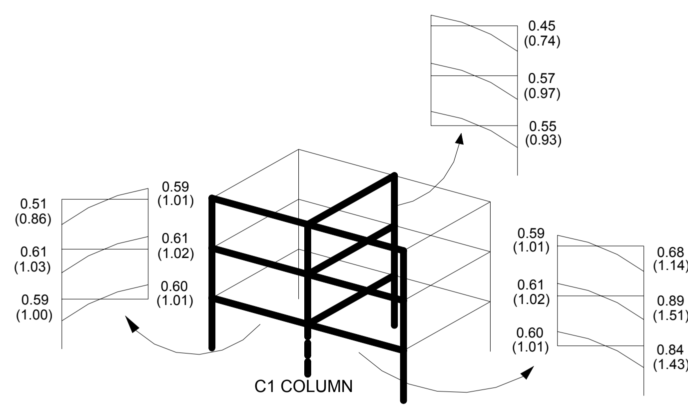
Figure 5.9 Linear dynamic analysis for 144 psf

Under a load of 144 and 252 psf, deflections (Table 5.7 and 5.8) from the linear static analysis including dynamic factors (α=2.0) were very close to those from a linear dynamic analysis after removing each column. This dynamic factor in linear static analysis was obtained to have the same deflection level as that from linear dynamic analysis. Therefore, the dynamic factor of 2.0 used in the GSA and UFC guidelines would be acceptable for linear static analysis for this structure.

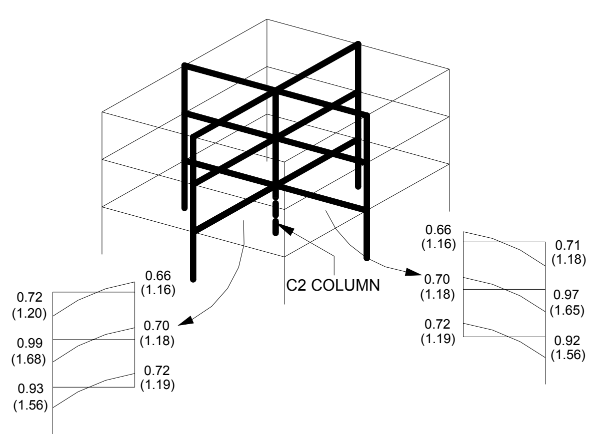
Demand capacity ratios (DCR), a ratio of force demand to capacity strength, at the far end of the beams supported by the removed column are shown in Fig 5.10 (a) - (c) for each removal case. Two values are shown at each end of beams. First values are obtained when a load of 144 psf is applied. Second values in parentheses represent DCR when a load of 252 psf is applied. Effective frames (those carrying loads from the removed column) are shown by thick solid lines. Under the design load of 144 psf, DCR values are smaller than 1.0. These small DCR values indicate there are sufficient capacities in elements after a column is removed. For a load of 252 psf, much larger DCR values are obtained. Although DCR values are less than 2.0 (progressive collapse expected for DCR more than 2.0 based on the GSA guideline), plastic hinges in beams and columns may be propagated. Moreover, axial-flexural plastic hinge properties are not considered in the linear analysis. In order to observe actual behavior of the structure, nonlinear analyses are required. In the following sections, nonlinear static and dynamic analyses are described.



(a) Corner column removal



(b) Edge column removal



(c) Interior column removal

Figure 5.10 DCR values at adjacent beam ends

5.2.3 Nonlinear analysis for a robust structure

To determine the nonlinear response of the frame, plastic hinges for moment-rotation characteristics and for shear demands are needed. The material properties and geometry of sections determine the moment-rotation response.

5.2.3.1 Material properties

Material properties of the concrete and steel are shown in Figs 5.11 and 5.12. The concrete strength was 4 ksi and steel yield was at 60 ksi. A strength increase factor of 1.25 was used for each material to take into account gain in concrete strength with time, concrete strength in situ, and steel strength in excess of ASTM requirements as well as strain hardening.

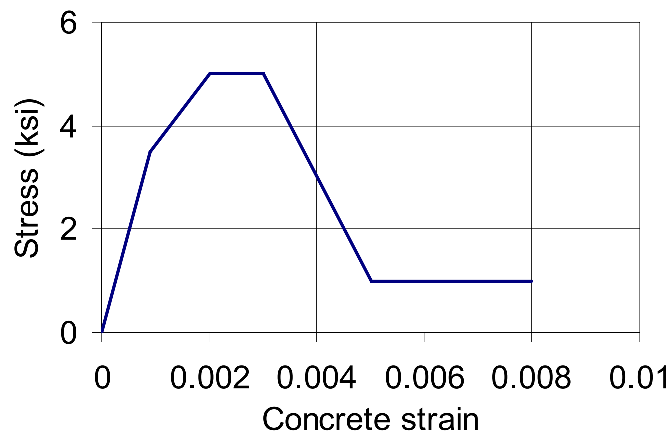


Figure 5.11 Material property in concrete

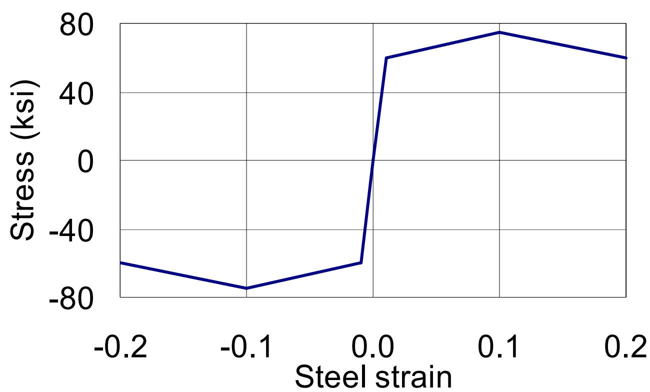


Figure 5.12 Material property in steel

Tension capacity for the concrete was excluded in the analysis because behavior in the inelastic range is expected, and cracking is extensive as ultimate strength of elements is reached.

5.2.3.2 Plastic hinge properties

Critical sections of the beams are shown in Fig 5.13. Beams had different cross-sections at exterior perimeter and interior locations, and details for four different sections are shown in Fig 5.14. Yield and ultimate strength were calculated for each beam including slab reinforcement shown in Fig 5.14.

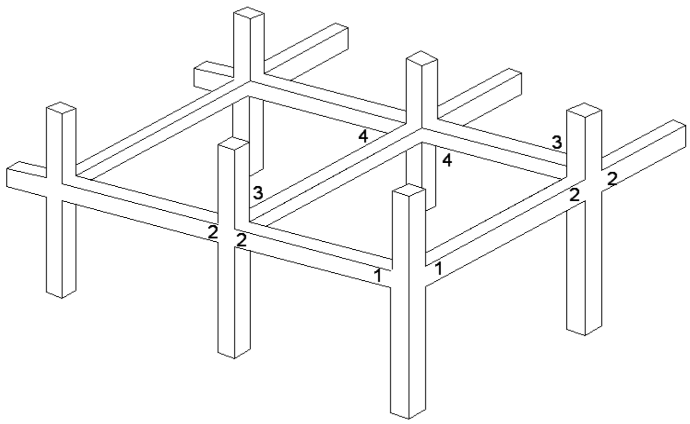


Figure 5.13 beam elements of the modeled structure

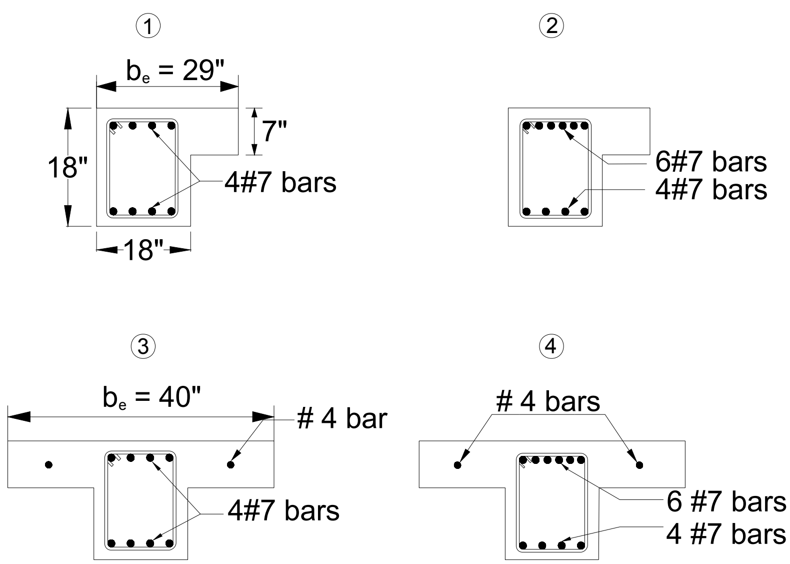


Figure 5.14 Reinforcement details in beams

Given the reinforcement shown in Fig 5.14, moment capacities for the exterior and interior beams and columns at exterior and interior perimeters were calculated and are summarized in Table 5.9. Capacities shown in Table 5.9 do not include axial load effect.

Table 5.9 Moment capacities for beams and columns

Location	Positive moment (k-in)	Negative moment (k-in)
1	2695	2586
2	2586	3768
3	2790	2983
4	2792	4150
Center of a beam	2611	2580
Exterior column	1962 k-in	
Interior column	4076 k-in	

Although beams near a support are subjected to negative moment under gravity loads, positive moment in Table 5.9 is also shown because moment reversals may occur when a supporting element is removed. Anchorage slip in beams and columns is assumed to be prevented because sufficient embedment lengths, as shown in Fig 5.6, are provided in a robust structure.

Axial force-moment interaction in beam hinge properties is considered to capture actual capacity of elements that carry axial forces. Rigid joint elements at each end of beams are modeled. When axial forces (tension or compression) are applied, moment capacity in a beam element changes as shown in Fig 5.15.

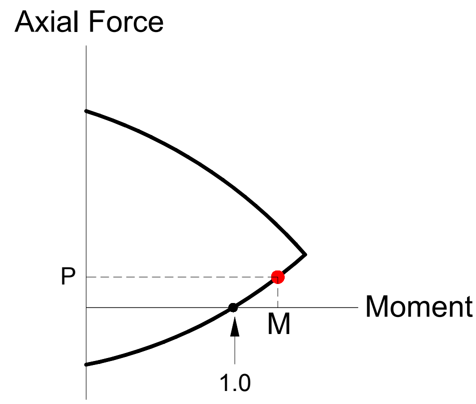


Figure 5.15 Axial force-moment interaction at beam and column hinges

When plastic hinges develop in members under compression or tension, ductility of beam elements is determined from the rotational capacity at that hinge. Rotational limits are discussed in 5.2.3.4.

Torsional rigidity of beams is assumed to be reduced to half of elastic torsional rigidity due to cracking expected under severe loading conditions. The current version of Perform-Collapse does not include torsional hinges so a modified stiffness was used.

Shear hinges at both ends and the center of beams and columns are established to determine if shear demand exceeds member capacity. Shear capacity for beams and columns is based on a constant shear stress over the depth of the member reflecting the contribution of both concrete and transverse reinforcement.

When column shear demand exceeds shear capacity, the member is considered to have failed. Although shear failures occur in a beam, it is assumed that that beam contains some residual shear stress (20% of the peak strength for beams controlled by shear) provided in FEMA 356 and adjacent slabs provide an alternate load path until beam or slab rotational limits are exhausted.

5.2.3.3 Flexural Design Capacities of Beams

In order to check if reinforcement provided in beams is suitable for a robust structure, flexural demands considering load factors are compared with capacities of each beam. In a robust frame structure, frame 2 (from Fig 5.7) is shown in Fig 5.16 to compare flexural capacities.

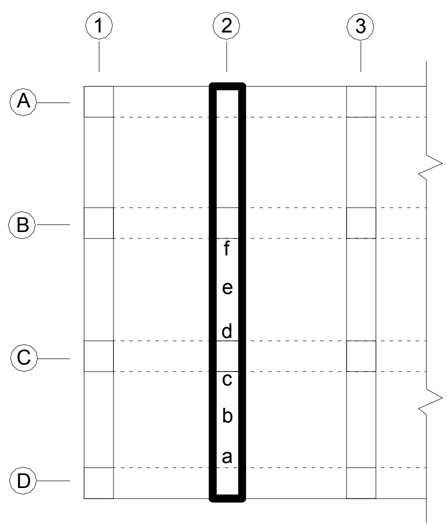


Figure 5.16 Frame 2 for flexure calculation

Flexural demands for the load combinations including dead, live, earthquake, and wind loads are calculated at each location. Critical values (M_u) from the load combinations are calculated and compared with M_u* from the load combinations based on analytical results in Table 5.10. Beam reinforcement shown in Fig 5.4 were established based on M_u values. Therefore, 6-#7 bars at location (c), (d), and (f) and 4-#7bars at location (a) and (b) were provided in the original design. Positive moment regions at location (b) and (e) were controlled by a load combination of dead and live loads. Negative moments at other locations were controlled by a load combination including dead, live, and earthquake loads. Ratios for the two values are very similar. Therefore, the structure is appropriately reinforced based on the load combinations.

Table 5.10 Flexure based on load combinations

Location	M_u (k-in) Original Design	M_u* (k-in) Perform-Collapse	M_u/ M_u*	Provided reinforcement
a	-1554	-1776	0.88	4-#7
b	1652	1386	1.19	4-#7
c	-2656	-2681	0.99	6-#7
d	-2552	-2492	1.02	6-#7
e	980	1146	0.86	3-#7
f	-2552	-2492	1.02	6-#7

However, flexural capacities in Table 5.9 show larger values at the same location than those in Table 5.10 because capacities of beams in Table 5.9 include strength increase factor of 1.25 for concrete and steel material and inclusion of slab reinforcements.

5.2.3.4 Rotational limit

For beam rotational limits, limit-state criteria from the GSA and UFC guidelines were used. Table 5.11 illustrates acceptance criteria provided by the GSA and UFC guidelines. Table 5.11 shows values for members acting as a tension membrane (catenary action), but response under catenary action was not considered in this study.

FEMA 356 [6] (for rehabilitation of structures in seismic zones) was used for the rotational limits for columns as indicated in the UFC guidelines. Detailed hinge properties for each component are given in Table 5.12 and represent values from the GSA, UFC and FEMA documents.

Actual moment-rotation curve used in hinges is shown in Fig 5.17. The rotational limit of collapse prevention (CP) for FEMA 356 is shown as point C in Fig 5.17.

Table 5.11 Acceptance criteria used

Component	GSA (rad)	UFC (rad)	FEMA 356 (rad)	
			C+	E
RC beam and slabs **with** shear reinforcing (**Without** tension membrane)	0.105	0.105	0.025	0.05
RC beam and slabs **without** shear reinforcing (**Without** tension membrane)		0.053	0.02	0.03
RC Frame action	0.035	-	-	-
RC beam **with** tension membrane	0.21	0.35	-	-
RC two way slab **with** tension membrane	0.21	0.35	-	-
RC column (Tension controlled)	0.105	1*	0.015	0.025
RC column (Compression controlled)	1*	1*	0.015	0.025
Sidesway for RC frame	H/25×	-	-	-

* Ductility in terms of φ_u / φ_y

× H is height of the vertical member

\+ Collapse Prevention (CP)

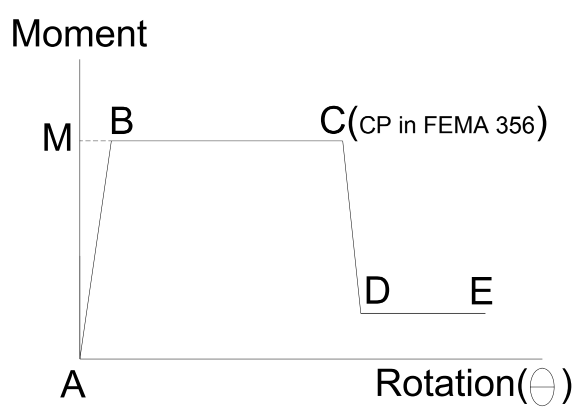


Figure 5.17 Moment-rotation behavior at hinges [4]

Table 5.12 Hinge properties used in Perform-Collapse program

Component	A		B		C		D		E	
	M	θ	M	θ	M	θ	M	θ	M	θ
Beam	1	0	1	0	1	0.035	0.2	0.036	0.2	0.053
Column	1	0	1	0	1	0.015	0.2	0.016	0.2	0.025
Slab	Maximum rotation limit = 0.105 radian based on GSA									
M is relative moment to yield moment θ : rotation (radians)										

In hinge properties for beams and columns, a normalized moment ratio of 1 indicates ultimate strength equal to yield strength. Rotation of 0.035 radian for beams was considered up to C (Collapse prevention level in FEMA 356) as the GSA guideline recommended for RC frame action. Maximum rotation of 0.053 radian was used based on the UFC guideline. Slab rotational limit used 0.105 radian based on both guidelines. As the UFC guideline suggests, column rotational limit used acceptance criteria from the FEMA document. Residual strength of 20% of the original strength was considered based on FEMA 356.

5.2.3.5 Dynamic factor and deflection from analytical results

Corner (D1), edge (C1), and interior (C2) columns as shown in Fig 5.7 were removed to investigate nonlinear behavior of the concrete structure. Nonlinear material properties for concrete and steel, as shown in Figs 5.11 and 5.12, were defined before running the program. Hinge properties of beams and columns for flexure and shear were also specified. Axial load capacity in a column above a removed location was based on the maximum tension capacity that was represented by the yield strength of the longitudinal column reinforcement. Large deflection (but not catenary action) in beams and slabs was considered.

In Table 5.13, deflections at the removed locations as well as corresponding dynamic factors for each removed column case are summarized.

Table 5.13 Deflection and dynamic factor for 144 psf

Removed location	Nonlinear Analysis	Load	Deflection (in)
Corner Column (D1)	Static*	***2.00*** x 144 psf	*2.50*
	Dynamic	144 psf	*2.48*
Edge Column (C1)	Static*	***1.86*** x 144 psf	*3.16*
	Dynamic	144 psf	*3.15*
Interior Column (C2)	Static*	***1.45*** x 144 psf	*6.60*
	Dynamic	144 psf	*6.60*

* With dynamic factor

Dynamic factors are calculated from nonlinear static analyses by incrementing loads on a structure until the energy balance between external work and internal energy is reached.

The applied load (144 psf on the floor and 115 psf on the roof) represents the actual loading situation. Dynamic factors calculated from nonlinear static analyses range from 1.45 to 2.0. Analytical results from nonlinear static analyses including a dynamic factor match well with those from nonlinear dynamic analyses.

When a corner column is removed for 144 psf, the structure remains in the elastic range of behavior. The dynamic factor (α =2.0) for 144 psf is the same factor used in the GSA and UFC guidelines. The structure can be interpreted to be strong enough to sustain the effects produced by loss of a corner column for the applied load. After removal of an edge column, if a gravity load with a dynamic factor (1.86) is applied on the remaining structure, some elements are in the inelastic range of response. When an interior column is removed, the smallest dynamic factor (1.45) and largest deflection at a removed location are obtained. As effective areas increase after removal of a column, dynamic factors decrease and deflections increase.

The robust structure successfully resisted additional dynamic load effects by removal of a column, and progressive collapse in a robust structure was not expected. Dynamic factors representing dynamic effects from removing a column were smaller than 2.0 regulated in the GSA and UFC guidelines. For the load (Dead load+ 0.25 Live load), a dynamic factor suggested by the GSA and UFC guidelines can be considered as an upper bound.

Dynamic factors and deflection curves for nonlinear static analysis are shown in Fig 5.18. Fig 5.19 compares deflections from dynamic analysis with those from static analysis including a dynamic factor.

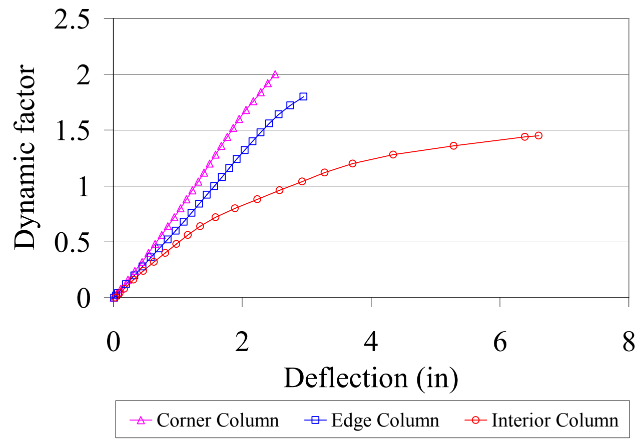


Figure 5.18 Nonlinear static analysis results at design load (144 psf)

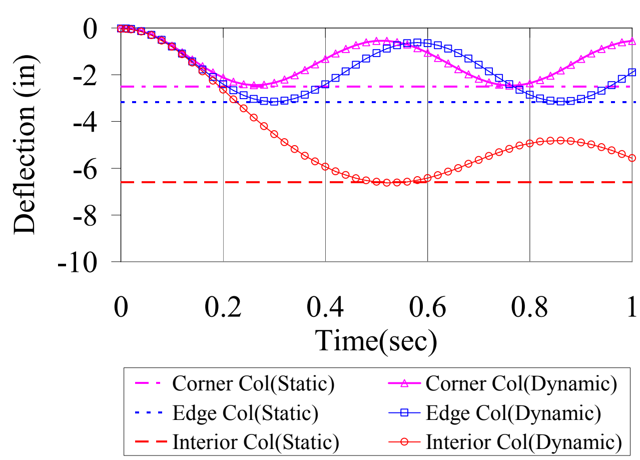


Figure 5.19 Nonlinear dynamic analysis results at design load (144 psf)

5.2.3.6 Behavior of structural members

- Corner column removal

Moment demand to capacity ratios are calculated and shown in Fig 5.20. Plastic hinges were not developed after a corner column was removed.

Axial force-moment interaction diagrams are plotted in Fig 5.21 at far ends of beams (location 1 in Fig 5.20) and corner beams (location 2). Column interaction diagram for a column at the third floor (location 3 in Fig 5.22) is plotted to investigate column failure by increase of bending.

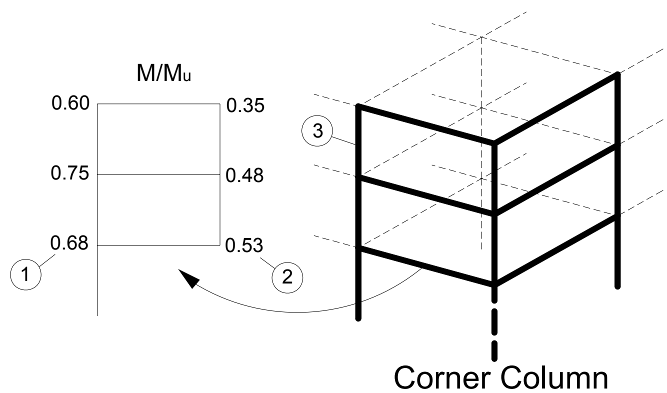
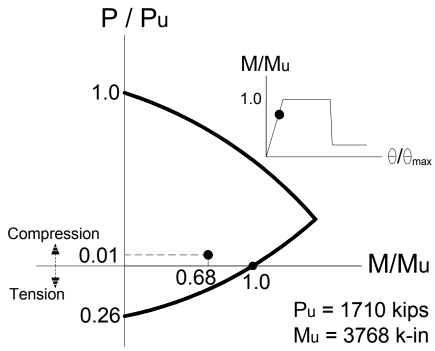
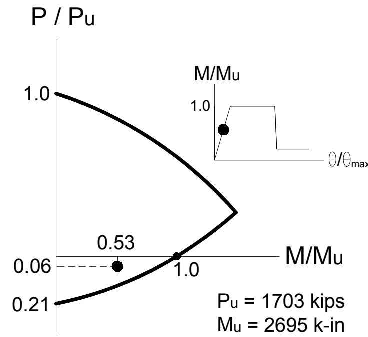


Figure 5.20 Observed locations when a corner column is removed

Each diagram shows the defined nominal interaction curve as a thick solid line. The moment-rotation relationship is normalized based on the maximum moments and maximum allowable rotations for members are shown in each plot. Moment capacity (M_u) was determined without considering the effect of axial force. Normalized values are obtained from results after a column is removed.



(a) Location 1



(b) Location 2

Figure 5.21 Axial force-moment interaction at beam ends

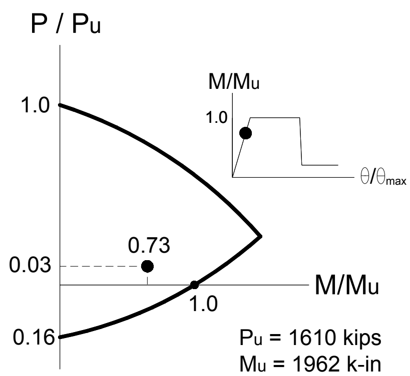


Figure 5.22 Axial force-moment interaction at a column (location 3)

Analytical results show all beams and columns in the elastic region. There are sufficient capacities to carry additional loads.

After a column is removed, additional loads cause in-plane tension forces at the column removed location and in-plane compression forces at the restraint boundaries. This is explained in Fig 5.23. When a load is applied in unrestrained beams or slabs, lateral expansion is observed as shown in Fig 5.23. If restraints are provided, compression forces are provided and these forces enhance behavior of the members [32]. Typically, these compression forces are ignored and assumed to be zero. However, enhancement provided by compression forces can provide slabs and beams with carrying more loads. These compression forces at the boundaries and tension forces at loading points are reflected in analytical results.

The interaction diagram at location 1 in Fig 5.21(a) shows the level of compression restraints provided by adjacent members. Tension forces generated at location 2 reduce the capacity, as shown in Fig 5.21(b).

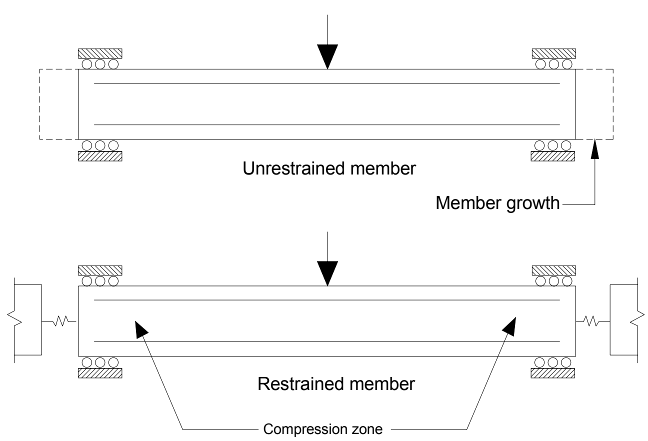


Figure 5.23 Restrained member [32]

Shear demands along beams and columns were approximately 18-20% of the capacity. The column above the removed column was subjected to tension of approximately 10% of its capacity. Slab rotational limits reached 20% of their maximum capacity. Therefore, the structure has sufficient capacity to carry larger loads.

- Edge column removal

Plastic hinges formed at ends of beams and columns in Fig 5.24 when an exterior column was removed. Three effective frames are divided to show ratios of demand to capacity.

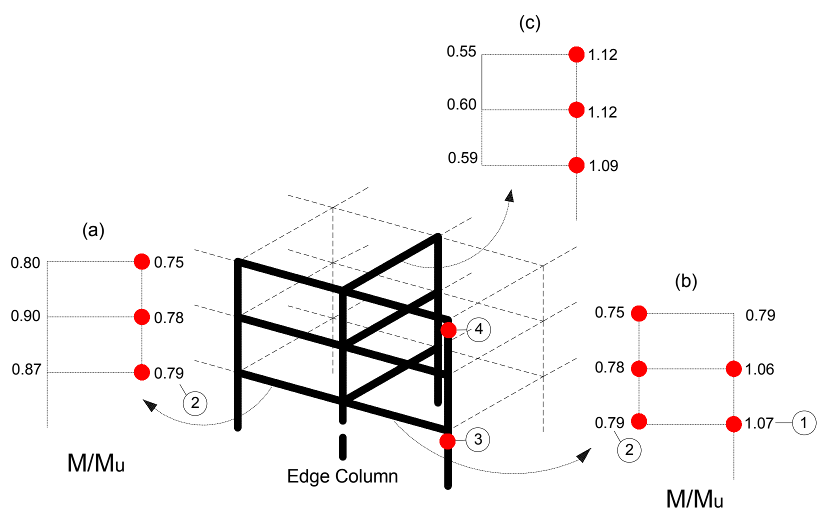
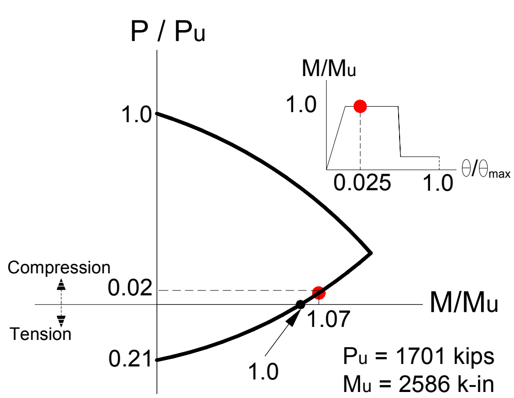


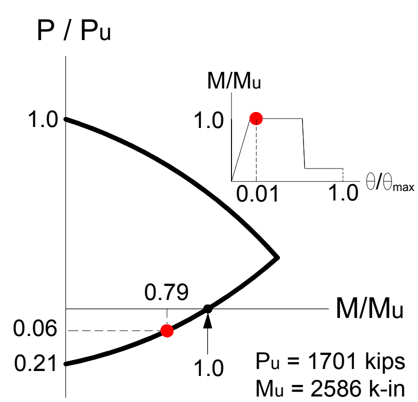
Figure 5.24 M/M_u at hinge locations when an edge column is removed

After removal of a column, increased capacity due to compression at the beam end (location 1 in Fig 5.25) was observed. The increased strength is also shown in Fig 5.25(a).

Although beam capacity was not reached at the ends nearest to the removed column (location 2), plastic hinges formed as shown in Fig 5.25 [frame (a) and (b)]. The effect of tension on the beam capacity is shown in Fig 5.25(b). Far ends of the beams in frame (c) shows plastic hinges with the moments reflecting the effects of compression on the beams. Plastic rotations are added in each plot to describe there is sufficient rotational capacity.



(a) Location 1

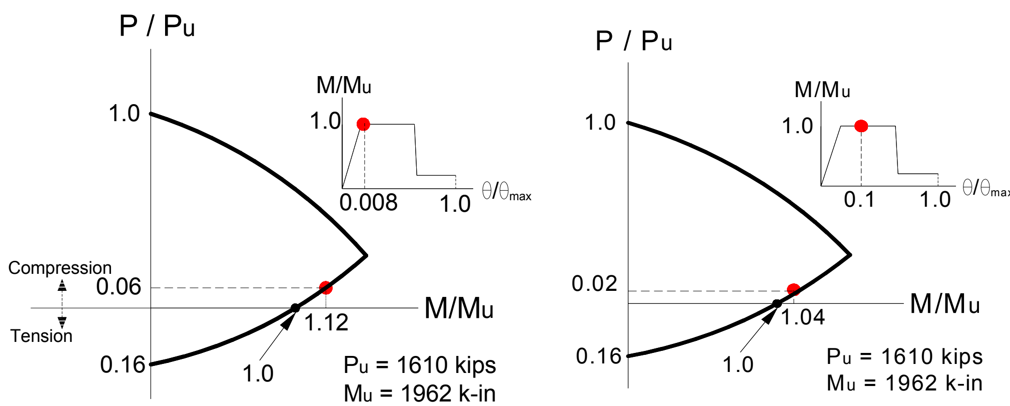


(b) Location 2

Figure 5.25 Axial force-moment interaction at beam ends

Columns at corner regions (locations 3 and 4 in Fig 5.24) exhibit plastic hinges on the first and third floors. Column interaction diagrams showing plastic hinges are plotted in Fig 5.26. Plastic rotation at location 4 is much larger than that at location 3. When the adjacent beams and slabs deflect, a column at the third floor controlled by bending does not have sufficient rotational restraints, compared with that at the first floor. Tension force at a column immediately above the failed column is only 15% of the tension capacity that the column can carry. Tension failure in that column is not expected.

Shear failure in beams and columns was not observed. Slab rotation reached about 21% of the rotational capacity regulated in the GSA and UFC guidelines.



(a) Location 3 (b) Location 4

Figure 5.26 Axial force-moment interaction at each column

- Interior column removal

When an interior column was removed, a number of plastic hinges at beams and columns were developed as shown in Fig 5.27.

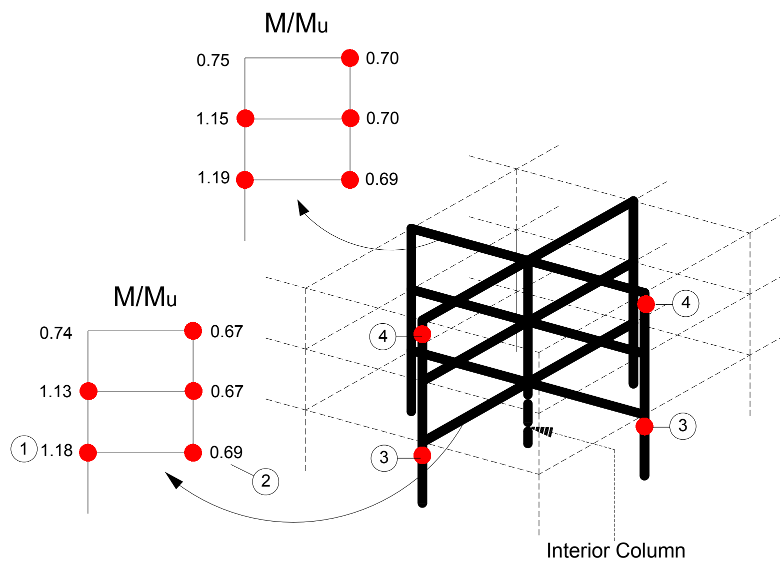
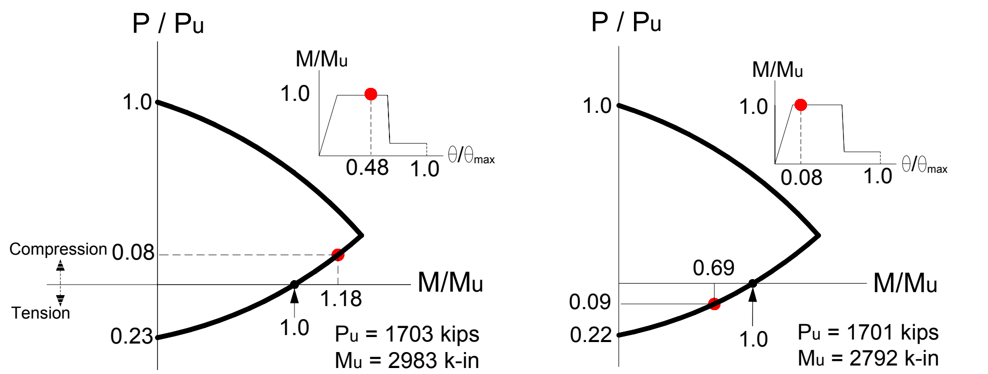


Figure 5.27 Hinge locations when an interior column is removed

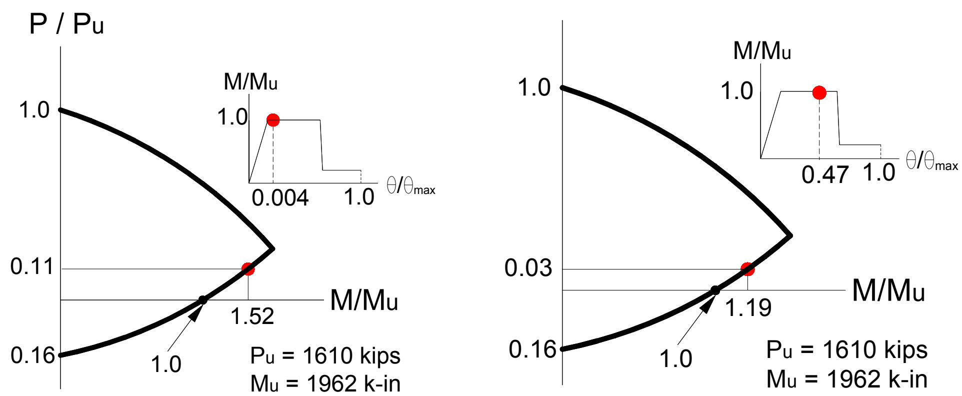
Plastic hinges formed in beam ends are shown in the axial force-moment diagram at each location (Fig 5.28). Axial compressive force at location 1 enhances the beam capacity and the plastic hinge rotation is 48% of the beam rotational limit (0.053 based on the UFC guideline). Beam capacity at location 2 decreases due to tension force in Fig 5.28(b). Although plastic hinges formed at each end of the beam, there are sufficient rotational capacities to carry additional loads.

Columns at the exterior perimeter showed plastic hinges with small rotation at the first floor and much larger rotation at the third floor as shown in Fig 5.29. Interior columns were stronger (8-#9bars) than columns (8-#6bars) at the exterior perimeter. The column at location 3 carries a larger axial force that enables the column strength to increase up to 50% in Fig 5.29(a). On the contrary, the column at location 4 is controlled by bending that requires much larger rotation in Fig 5.29(b).



(a) Location 1 (b) Location 2

Figure 5.28 Axial force-moment interaction at beam ends



(a) Location 3 (b) Location 4

Figure 5.29 Axial force-moment interaction at each column

Tension force in the column above the removed column was 5% of the tension capacity in that column because all floors deflect at the same time, so there is not much tension developed between floors.

Shear capacity was not exceeded at beams and columns, and only 46% of the slab rotational capacity was reached. Therefore, the interior slab panels and beams have adequate capacities to carry larger loads.

5.2.4 Summary of a robust structure

A robust structure with good detailing and sufficient ductility was analyzed using linear and nonlinear analyses.

In a linear analysis, dynamic effects resulting from removal of a column produced a dynamic factor of 2.0. Under the service load condition, the structure remained elastic when a corner column was removed, and a dynamic factor of 2.0 was obtained. When an edge and an interior column were removed, dynamic factors smaller than 2.0 (1.45 and 1.86) were observed because areas affected by removal of a column increased. A structural system with a smaller dynamic factor indicates that structural members have some damage and load redistribution occurs. These dynamic factors represent how loads are distributed as the magnitude of the gravity loads increases. It should be remembered that there are uncertainties in locations of the hazard, size of the impact, and size of the damage areas. For design, it may not be conservative to conclude that dynamic factors smaller than those recommended in the GSA and UFC guidelines should be used. A dynamic factor of 2.0 is reasonable considering dynamic effects for removal of columns in a robust structure.

Continuous reinforcements provided in connection regions of the robust structure prevented abrupt failure in members when load was redistributed after removal of a column. Removal of a corner column did not generate any failure because sufficient anchorage was provided. When an edge and interior columns were removed, plastic hinges at beams nearest to the column-removed location occurred. However, sufficient ductility prevented failure at those sections. Without continuous reinforcement, abrupt local failure at connection regions can be expected. Structural system failure depends on the how well alternate load paths are provided after the local failure occurs. A structure without structural integrity and good detailing is discussed in Chapter 7.

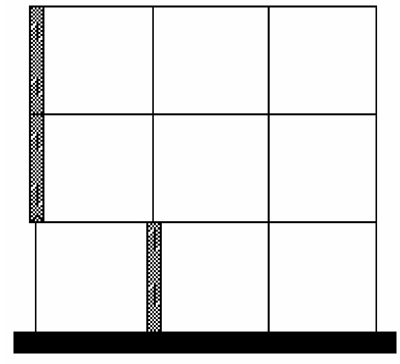
CHAPTER 6

Deficient Aspects in Structures

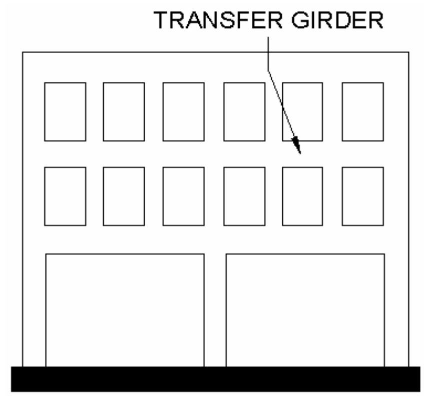
6.1 STRUCTURAL CONFIGURATIONS AT RISK

Examples of irregular structures at risk when abnormal loads are applied are shown in Fig 6.1. FEMA 273/356 indicates vulnerability of irregular structures under seismic loads unless these structures are carefully detailed. Irregular structures pose similar risks for progressive collapse.

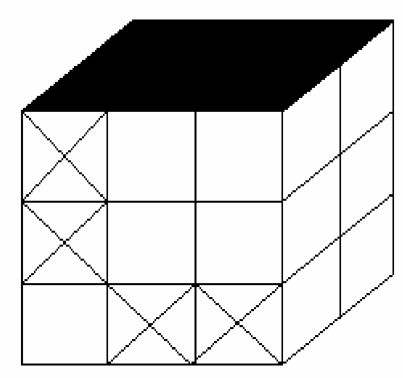
In the case of (A) in Fig 6.1, an abnormal loading event on the irregular first floor may cause damage to a critical element supporting a large area of upper floors. Case (B) is a simplified version of the Murrah Federal building in Oklahoma City. This configuration is very common in office buildings in the US. Transfer girders distribute vertical loads from a number of columns supporting the upper floors to large second floor girder that is supported by only a few columns. Severe damage or failure of even one of the columns on the first floor can result in catastrophic collapse if alternate load paths are not available to redistribute the gravity loads. Case (C) has quite similar characteristics to case (A).



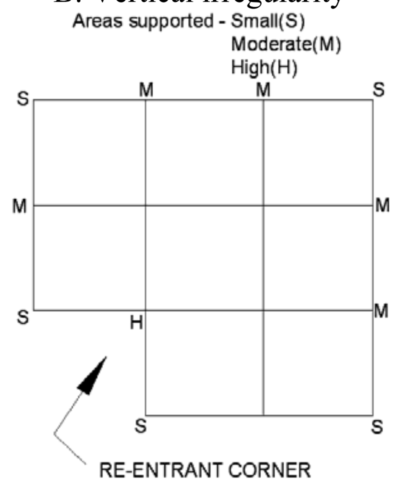
A.Vertical irregularity



B. Vertical irregularity



C. In-plane discontinuity



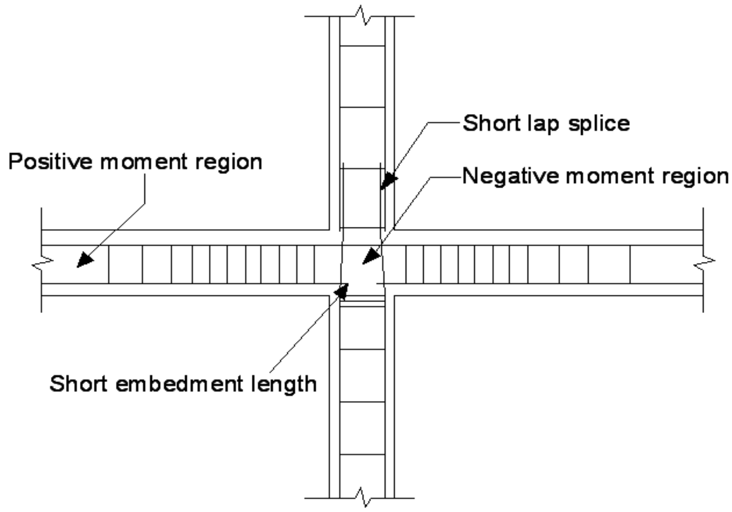
D. Re-entrant corner

Figure 6.1 Irregularity in plan and elevation [6]

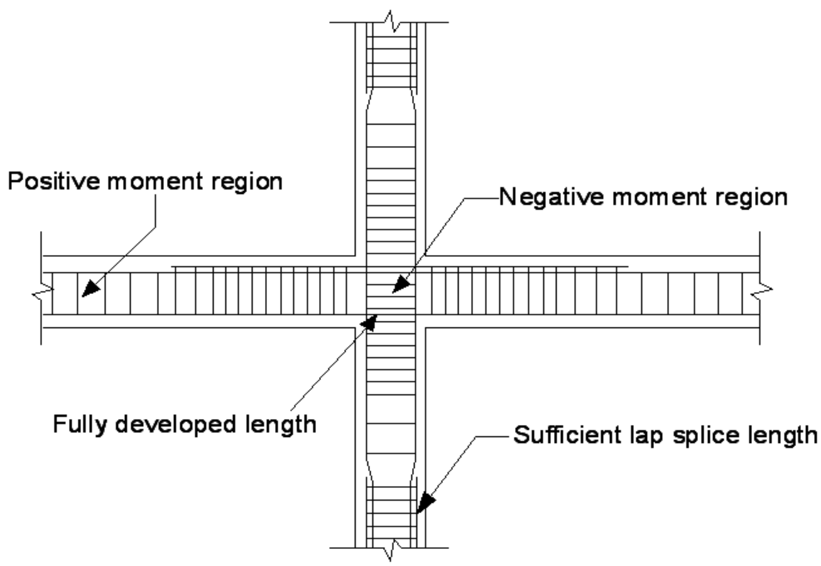
Importance of exposure to abnormal loadings is illustrated by case (D). Exterior frames are more exposed to external threats compared with interior frames. Damage to corner columns may cause local collapse of the structure at the corner. However, if an interior column is damaged, a structure may collapse unless adjacent member capacities are sufficient. Therefore, a re-entrant corner has two deficiencies: exterior exposure to a threat and larger gravity load distribution on the column at the reentrant corner. Blast design loads may be amplified because the adjacent walls direct blast effects toward the re-entrant corner.

6.2 STRUCTURE WITH INADEQUATE DETAILS

Existing concrete structures may have insufficient alternative load paths, compared with well-detailed concrete structures built to current standards. Concrete structures built before the 1980s do not have structural integrity reinforcement now required in the ACI code. As a result, they do not have sufficient flexural capacity to develop alternative load paths, especially when compared with concrete structures built in a seismic zone. Reinforcement details of many typical pre-1980s concrete moment frames are shown in Fig 6.2 (a). A robust structure with well detailed reinforcement is also shown for comparison in Fig 6.2 (b). In the following sections, three potentially critical deficiencies can be identified in buildings constructed in the 1980s: reinforcement continuity, shear reinforcement requirements, and lap splices in a column.



(a) Typical concrete frame in the 1960s



(b) Robust structure built to current standards [31]

Figure 6.2 Details of deficient and robust concrete frame structures

6.2.1 Beam reinforcement continuity

In order to develop alternate load paths, continuous reinforcement and sufficient anchorage for positive moment is needed between adjacent members. The lack of such continuity can be a critical factor in progressive collapse. Continuous bottom reinforcement can allow catenary action to develop if support provided by a critical column connecting spans is lost. Therefore, the connection where beams and columns intersect is one of the locations where continuity must be provided. Prior to the 1980s, short extensions of bottom beam bars in the ACI code were allowed. More stringent provisions for structures in seismic zones were established in the 1970s. However, there are many structures in the US with bottom (positive moment) beam bars extending into the support only 6 in. Such extensions are not sufficient to provide the tension capacity needed for catenary action when a column under the connection is removed [48]. As catenary action develops, negative moment at the face of the support shifts to positive moment and eventually the section can only carry tension through the top bars. If there are sufficient ties or stirrups in the beam, it may be possible to develop tension along the beam for catenary action, but it is likely to be developed only after large vertical displacements occur. This inability to efficiently develop catenary action may lead to catastrophic failure.

In ACI 318-89, a structural integrity requirement was added [1]. One-quarter or one-sixth of the reinforcement or at least two continuous bars had to be provided along exterior beams. This deficiency will be labeled "Inadequate Continuity".

Therefore, structures built after the 1990s following structural integrity and earthquake resistant detailing requirement have at least two continuous bottom beam bars through the column.

6.2.2 Shear reinforcement

Insufficient shear transverse reinforcement in Fig 6.2 (a) along structural elements can lead to the possibility of collapse. When the shear force distribution changes due to unexpected loss of a support, large shear forces not considered during the initial design may act on the beams. Therefore, concrete sections between widely spaced transverse reinforcement near the center of the original beam may be subjected to shear greater than capacity provided by the concrete. This may cause a brittle failure in the member and may trigger failure in adjacent members. This deficiency will be defined as "Inadequate Shear Capacity".

6.2.3 Column splices

Insufficient lap splice lengths in a column can also be a potential trigger of progressive collapse. In most cases, the column reinforcement will be designed using compression lap splice length requirements because the columns are assumed to carry only compression. Column splices are generally located at the bottom of the column.

Since 1989, the ACI code has required consideration of tension capacity at compression splices due to moments developed under wind and seismic loads. When a column in the first floor is removed because of an abnormal loading, the column above must support the suspended slabs and beams. It is unlikely that tension was considered in the original design. Tension lap splice lengths are longer than those for compression. Failure of a splice will eliminate any possibility of transferring loads supported by the removed column through “bridging” to upper floors that have not been damaged. This deficiency will be termed as a “Splice Deficiency”.

6.3 REHABILITATION OPTIONS

In FEMA 356 [6], rehabilitation of existing buildings in seismic zones is described in detail. Simplified retrofit schemes for small and regular structures and systematic retrofit schemes for many types of buildings are addressed. In some cases, additional or alternate lateral load resisting systems are recommended.

Improved detailing and better confinement will enhance member strength and ductility for moment and shear forces. Confinement improves development of tension forces in a member that has been designed only for compression or has not been designed for sufficient tension.

Although use of seismic detailing has been suggested as a means of mitigating the effects of progressive collapse, it may not be sufficient for existing structures with serious construction errors, old buildings lacking in structural integrity, or poor layout. The collapse of the Sampoong department store in Korea in 1995 is one example. During the construction of the building, the owner changed the original design plan to increase office spaces, built an additional floor level without authorization, and set up air-conditioning equipment on the roof. These expansions and additional loads were not considered in the original design [40]. As a result, the roof collapsed and fell on the floor below. A local collapse of that floor led to the collapse of the entire building in a short time. In this case, although good detailing might have helped, it would not have prevented the total collapse of the building due to disregard of building safety requirements. Collapse of the structure might have been avoided if alternative load paths had been provided to transfer loads from roof failures to other columns supporting the upper floor. Collapse of the Sampoong department is illustrated in Fig 6.3.

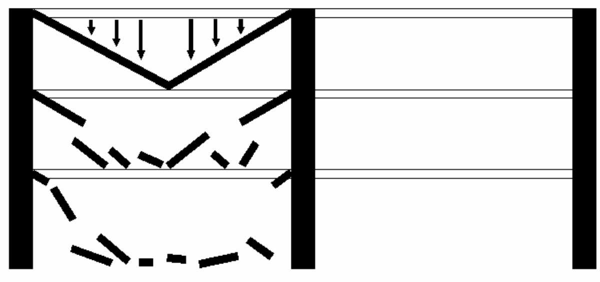


Figure 6.3 Progressive collapse disproportionate to a local collapse

A second case is the collapse of the Murrah Federal Building in Oklahoma City that was constructed in 1974. Although failure of the column located at the point of the explosion could not have been prevented, adjacent columns on the first floor might have limited the results of the explosion to local collapse [18, 19]. Fig 6.4 illustrates behavior when an important vertical element is badly damaged and no alternate load paths available (in this case, flexural capacity at ends of beams of beams)

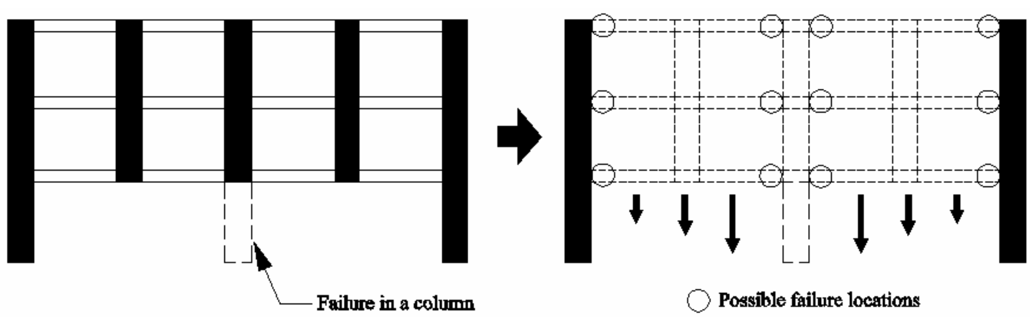


Figure 6.4 Structural collapse due to limited alternate load paths

Retrofit schemes for unexpected loading situations can be established based on the development of the alternative load paths or by providing specific local resistance.

6.4 ALTERNATIVE LOAD PATH METHOD (ALPM)

Modern structures having sufficient ductility, continuity and redundancy can develop alternate load paths in case abnormal loads are experienced. Sturctures can be modified to improve their ability to withstand unexpected loads. Structures built in seismic zones often may be improved by adding new or strengthening existing lateral force resisting systems. Similarly, gravity load carrying capacity may need to be augmented to reduce the risk of progressive collapse.

A structure should have sufficient capacity to transfer applied loads to the adjacent components. Alternate load paths can be provided in a retrofit scheme. One of the methods for rehabilitating a structure can be compartmentalized construction [19]. By providing many wall components, alternate load paths are available. A good example of a robust structure is a bearing wall panel structure where the loss of one wall limits damage to that compartment [19]. However, such compartmentalization can not be provided if large open areas are needed [19].

Fig 6.5 illustrates a robust structure after removal of a load bearing component. For existing structures, more walls can be established to ensure load redistribution. In addition, ductility in structural components is essential for sustaining and redistributing additional loads.

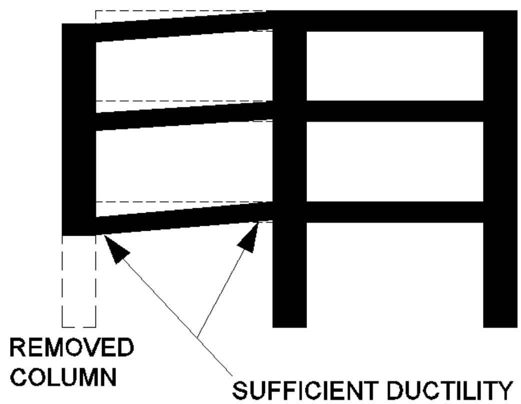


Figure 6.5 A robust structure

6.5 SPECIFIC LOCAL RESISTANCE METHOD (SLRM)

Another rehabilitation method is directly strengthening a specific load bearing element that may be at a location vulnerable to an external attack. Fiber composite materials can be used to wrap a column as shown in Fig 6.6 to provide strength and ductility sufficient to resist a specified external explosion for a certain level of threat. Wall components can also be protected using CFRP (Carbon Fiber

Reinforced Polymer) to minimize shattering debris inside of a structure as shown in Fig 6.7 [20].

Figure 6.6 Retrofitted column by CFRP [20]

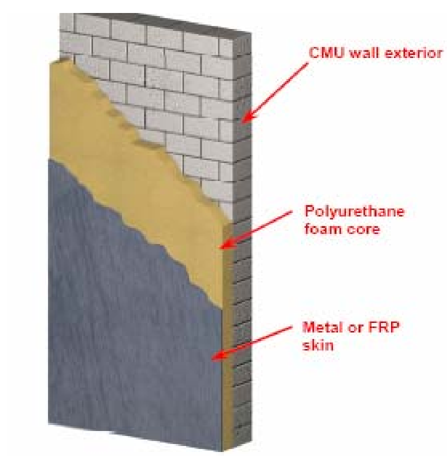


Figure 6.7 Retrofitted wall by CFRP material [20]

CHAPTER 7

Analysis of Structures with Deficiencies

The behavior of a robust structure was presented in Chapter 5. A robust structure is defined as one having sufficient strength, ductility, continuity, detailing, and structural integrity to withstand the effects of loss of a major element. A deficient structure is an existing structure designed using provisions that did not have the more stringent detailing and structural integrity requirements of current codes. Inadequate development of reinforcement near connections, insufficient shear capacity along beams, and inadequate lap splice capacity in columns are considered in this chapter.

The effect of a short embedment length of the bottom flexural reinforcement at a connection is considered. The cracking moment is assumed to be the maximum moment capacity for the positive bending at a connection to reflect a short embedment length. Insufficient transverse reinforcement for shear in a concrete beam is considered by assuming that only the concrete resists shear at the original mid-span of beams. It is not easy to define shear displacement limits for design when shear failure controls. Although shear resulting from column removal may cause a shear failure at some section of a beam, other beams and floor slabs are assumed to carry those forces by alternate load paths. FEMA 356 provides rotational capacities for a flexural member when shear force controls structural response. Beam and slab elements in the alternate load path can resist additional loads until their rotational capacities are exhausted. In general, the rotational limit of beams is exceeded before that of slabs [34, 46].

The effect of an inadequate lap splice length in a column is examined at columns located above the removed column. If the tension capacity of the column is exceeded and the surrounding floor is not able to carry a significant load increase, a load path that transfers forces to floors above the removed columns is no longer valid.

7.1 BACKGROUND OF THE MODELED STRUCTURE

The Perform-Collapse program [56] was used to study deficient structures. Information for the structure modeled in the program is illustrated below. The prototype structure taken from the PCA document was used. Reinforcement and dimensions are the same as the structure used in Chapter 5, except where deficiencies are incorporated in the models.

7.1.1 Material properties

Material properties for concrete and steel reinforcement are as shown in Figs 5.11 and 5.12.

7.1.2 Modeling of the prototype structure

The structure modeled is shown in Fig 7.1, and columns removed are identified. Design loads of 144 psf (floor) and 115 psf (roof) that represent actual loading conditions were applied on the floor.

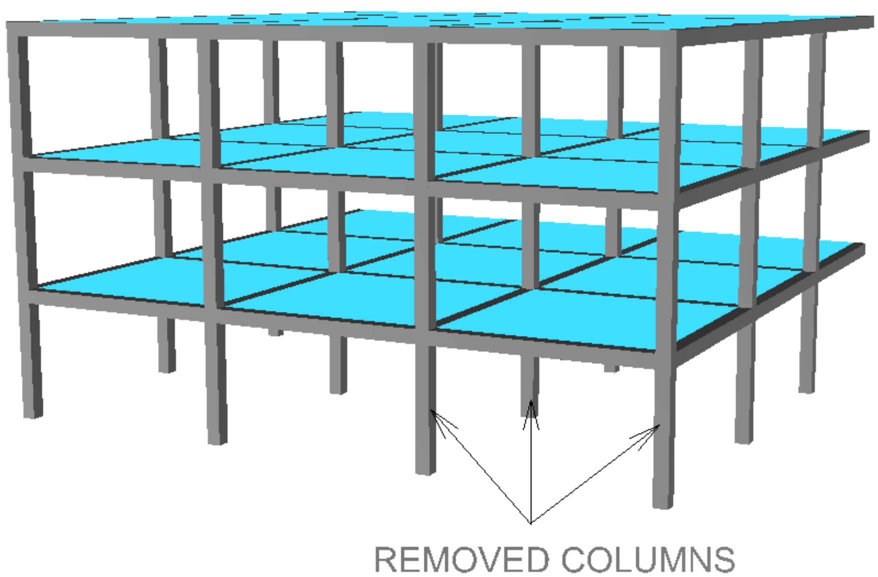


Figure 7.1 Removed columns at the first floor

Moment, shear, and axial capacities at beams and columns were defined in the program using the dimensions and reinforcement shown in Figs 5.1-5.5.

Deficiencies of short embedment length at bottom reinforcements of beams, insufficient shear capacity at a mid-span of a beam, and insufficient lap splice length along columns were considered as shown in Figs 7.2 and 7.3.

Deficiencies in a structure were reflected in capacity calculations. A typical embedment length of 6 inches through the connection area and axial tension deficiency at a column above the removed column location are shown in Fig 7.3.

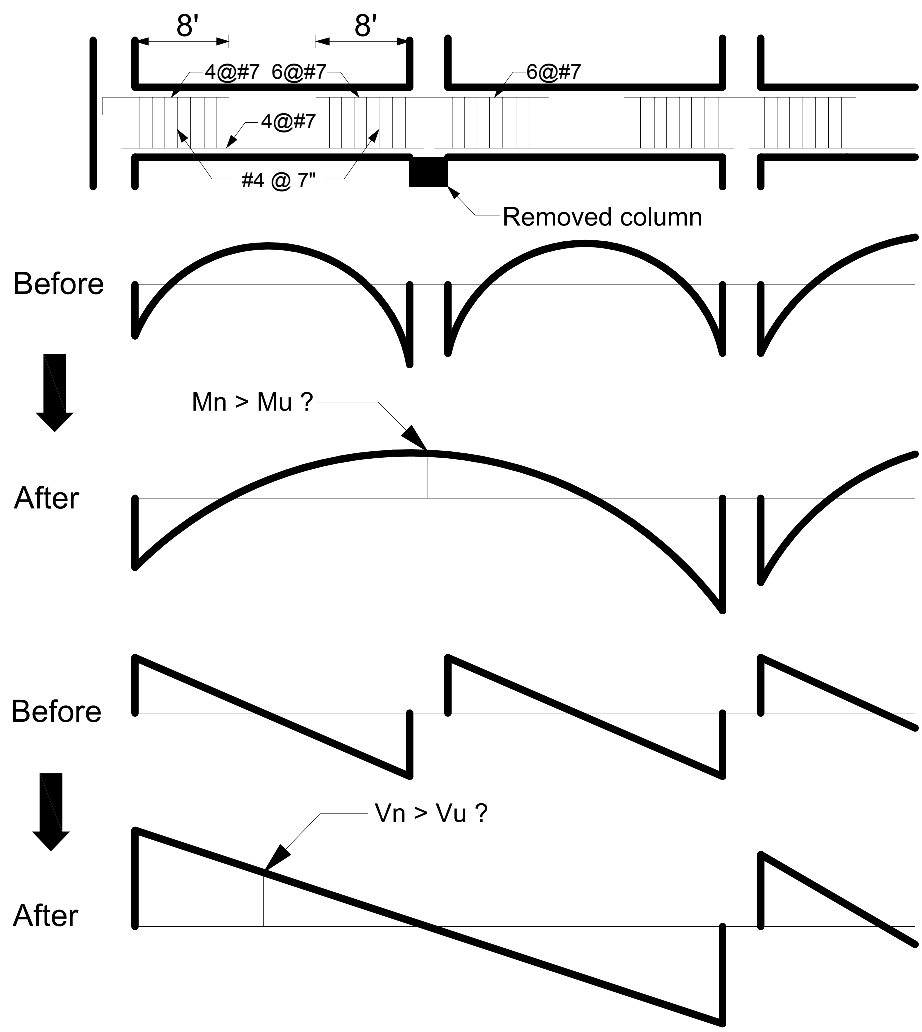
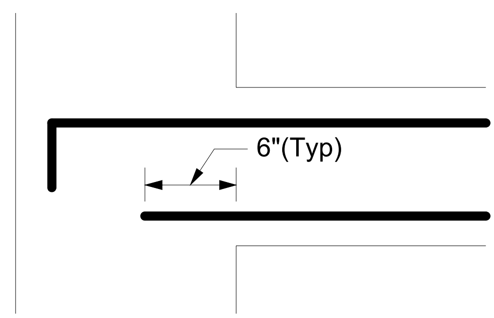
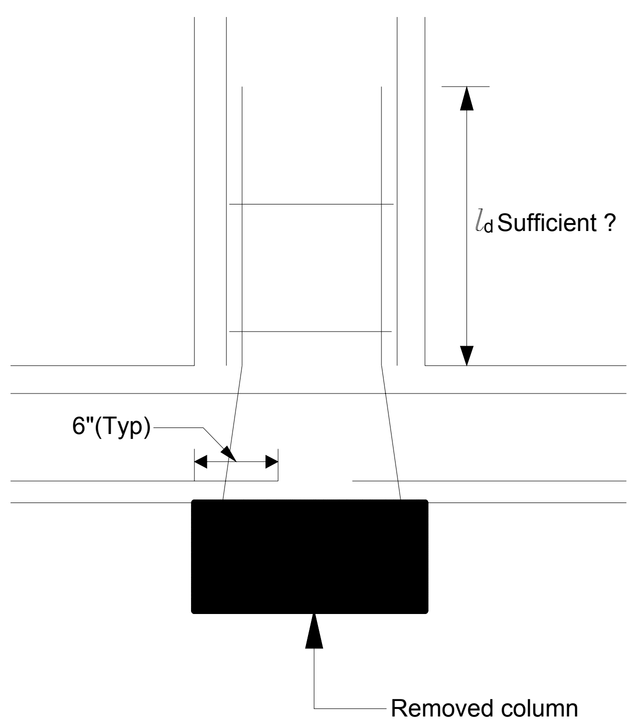


Figure 7.2 Deficiencies considered for moment and shear



(a) Typical embedment length at an exterior beam and slab



(b) Short development length at interior region

Figure 7.3 Deficiency of development length and lap splice length

Response characteristics at locations of inadequate continuity

Insufficient development length is considered at connection regions (locations 1, 2, 3, and 4 at Fig 7.4). The development length or anchorage of the bottom longitudinal reinforcement at the connection into the column is inadequate to allow the reinforcement to yield. This deficiency is reflected in calculating positive moment capacity at these locations. Therefore, cracking moment (M_{cr}) is considered as a maximum capacity that a concrete section can carry when a column is removed. After reaching the cracking moment, a residual stress of 1% of the cracking moment capacity is considered up to the rotational limit because the program does not allow zero strength.

The plastic rotation limit for a beam with the insufficient development length is 0.02 in FEMA 356 [6]. Although the cracking moment is reached, adjacent beams and slabs enable loads to redistribute, so large rotations (0.053 rad for the UFC guideline in Table 5.9) is assumed to be acceptable. The rotation is larger than that given in FEMA 356 for beams with inadequate reinforcement continuity. Cracking moment and plastic rotation in beams with inadequate development length is shown in Fig 7.5.

The same axial force-moment interaction and rotational limits as in a robust structure are used for all other sections, and large deflections are permitted.

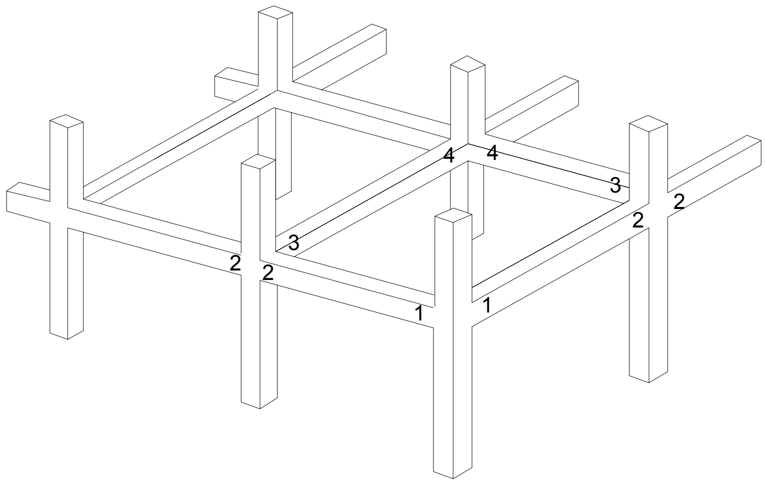


Figure 7.4 beam elements of the modeled structure

Table 7.1 Positive and negative moment capacities for beams shown in Fig 7.4

Location	Positive moment (k-in)	Negative moment (k-in)
1	533	2586
2	533	3768
3	533	2983
4	533	4150
Center of a beam	2611	533

M_{cr} = 533 k-in

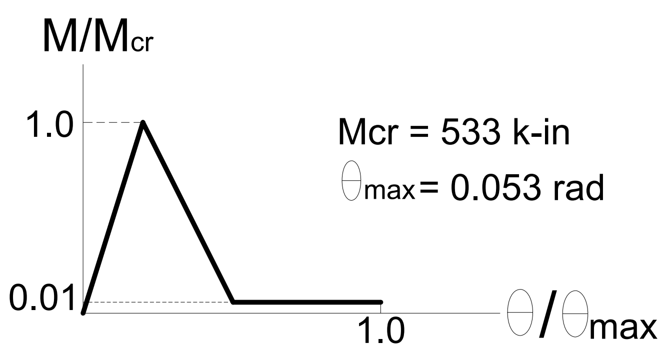


Figure 7.5 Moment-rotational relationship considering insufficient development

Response characteristics at locations of inadequate shear capacity

The shear force-displacement relationship near the mid-span of a beam is shown in Fig 7.6. Shear capacity at such a location is provided by the concrete, and there is little or no contribution from widely spaced transverse reinforcement. Shear failure may occur when the shear force distribution changes after a column is removed (Fig 7.2).

When shear demand exceeds shear capacity, a brittle failure occurs. For this study, it is assumed that a residual shear capacity up to 20% of the maximum shear capacity is maintained for beams controlled by shear as recommended in FEMA 356 [6]. Displacement limits for shear-controlled beams are not defined in FEMA or other documents. Instead, FEMA 356 provides rotational limit for beams controlled by shear. Using this rotational limit, the total displacement (Δ) including flexural and shear displacements at location of the shear deficiency to the clear span length (L) is considered. A shear force–displacement relationship is shown in Fig 7.6. Shear demand (V) is normalized for shear capacity (V_u).

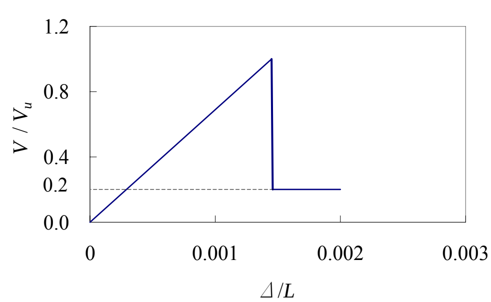


Figure 7.6 Shear force-displacement at mid-span

After shear failure occurs, load is redistributed to adjacent beams and slabs until plastic hinges in beams and slabs reach their rotational limits. Therefore, the structural behavior is governed by actions in adjacent beams and slabs.

Splice deficiency

Insufficient tension capacity at a column above the removed column may result in tension failure of the column as shown in Fig 7.7. Conservatively, tension capacity was provided by tension resistance from the longitudinal reinforcement in columns. Axial tension capacity for a column was assumed to be the yield strength of the reinforcing steel in the column. When the tension demand (P) reaches tension capacity (P_u) in a column, that column is assumed to no longer have any tension load-carrying capacity as shown in Fig 7.7.

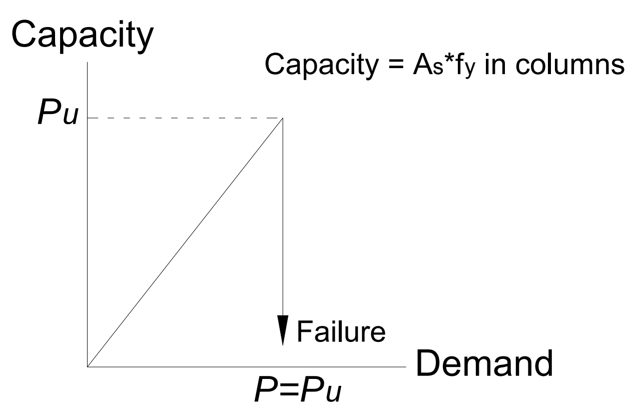


Figure 7.7 Comparison between tension capacity and demand in a column

Column bars are generally spliced just above the floor or beam as shown in Fig 7.3(b). Column lap splices that are designed for compression may not be sufficient to resist tension force and may fail.

Compression and tension splice lengths are compared in Table 7.2. Lengths were calculated based on #9 bars, a concrete strength of 4 ksi, and a steel strength of 60 ksi. Steel yield was considered to be the maximum allowable stress in the reinforcement.

Table 7.2 Comparison of lap splice length

Force	ACI 318 code			
	1951	1963	1971	2002
Compression	22.6 in	27.1in	34 in	34 in
Tension	22.6 in	40.6 in	54 in	54 in

Tension splices are much longer than compression splices as shown in Table 7.2. When the applied force changes from compression to tension, columns in old existing structures have approximately 40-60% of the required tension splice length. If a column above the removed column has 60% of the required tension splice length under tension force acting in that column, the maximum tension demand in that column is considered up to 60% of the capacity because a linear relationship between lap splice length and steel yield is assumed. Therefore, the maximum tension capacity at a column is adjusted for the actual splice length.

Based on Table 7.2, if tension demand in a column exceeds 40~60% of the required tension strength at yield, lap splice failure can be expected.

7.2 ANALYTICAL RESULTS FOR A DEFICIENT STRUCTURE

The structure was analyzed under design loads of 144 psf (floor) and 115 psf (roof). In Table 7.3, deflections and dynamic factors from each static and dynamic analysis are summarized.

Table 7.3 Deflection and dynamic factor for each column removed (144 psf)

Removed location	Nonlinear Analysis	Load	Deflection (in)
Corner Column	Static*	***1.72*** x 144 psf	*3.99*
	Dynamic	144 psf	*3.98*
Edge Column	Static*	***1.60*** x 144 psf	*5.59*
	Dynamic	144 psf	*5.59*
Interior Column	Static*	***1.52*** x 144 psf	*9.94*
	Dynamic	144 psf	*9.93*

* With dynamic factor

Dynamic factors ranged from 1.52-1.72. These values are below the dynamic factor of 2.0 in the GSA and UFC guidelines. At energy balance, dynamic deflections caused by removal of a column match nonlinear static deflections increased by the dynamic factor.

Deflections at the removed column locations increased as tributary areas of the floor affected by removal of a column became larger, but dynamic factors decreased as tributary area increased. It can be interpreted that a smaller dynamic factor should be considered to reach energy balance between external work due to dynamic effects and the internal energy of the remaining structure. Fig 7.8 shows deflections at the removed column locations for each static and dynamic analysis.

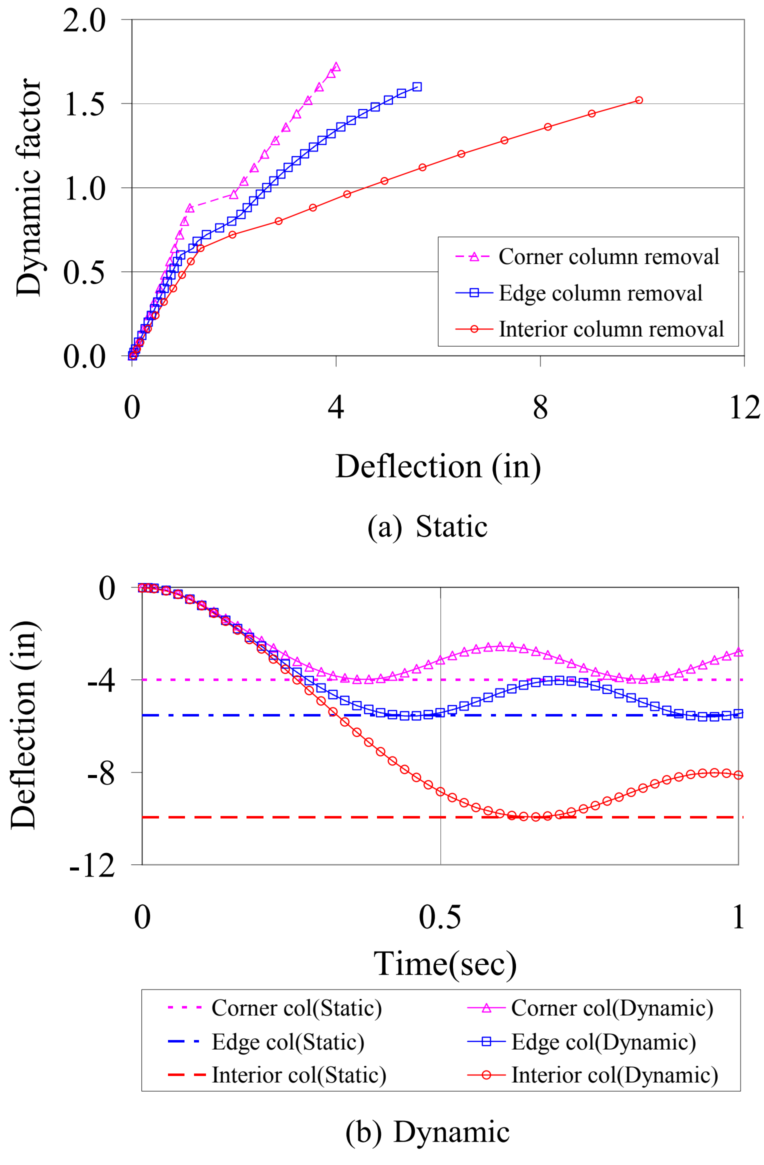


Figure 7.8 Analytical results from nonlinear static and dynamic analyses (144 psf)

Inelastic dynamic analyses are bounded by a straight line that represents deflection at the removed location from inelastic static analyses including a dynamic factor.

Static load-deflection curves show a sudden increase (kink) in deflection. This increase was the result of cracking (positive moment) at column-removed locations and redistribution of loads to other sections and elements.

Slabs did not reach their rotational capacity as defined by the GSA guideline. Although a large deflection at the interior location was computed, the structure was able to absorb sufficient energy for increased dynamic loads after removal of a column.

7.2.1 Detailed structural behavior

For each removed column, the moment-rotation behavior of spandrel and interior beams, shear behavior in beams, and tension behavior in columns were investigated.

7.2.1.1 Corner column

The locations at which moment, shear, and axial capacities are computed for removal of a column at a corner panel region are shown in Fig 7.9.

Flexure behavior

When a corner column is removed, each beam end at location 1 (Fig 7.9) is in negative bending. The capacity may be exceeded after removal of a column. A beam end adjacent to the corner column region is controlled by negative moment before a corner column is removed. After removal of a column, the moment configuration changes from negative to positive moment. A short embedment length as shown in 7.3(a) is assumed at this location. The cracking moment is assumed to be the maximum moment for the positive moment at that location.

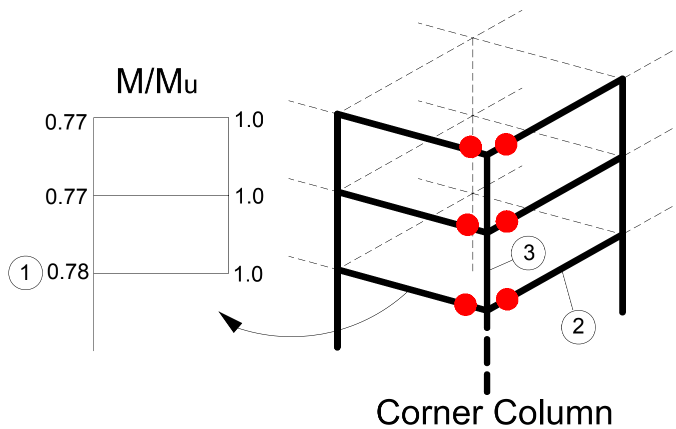


Figure 7.9 Locations studied when a corner column is removed

Although significant change in the moment configuration was observed, the far ends of the adjacent beam did not form plastic hinges. Ratios of demand to capacity were 0.77-0.78 at these locations (Fig 7.9). Axial load-moment interaction at location 1 is exhibited in Fig 7.10. Axial-moment interaction curve at location 1 as shown in Fig 7.10 indicates sufficient beam capacity to carry additional loads caused by removal of a column. A compression axial force was observed at location 1 due to expansion of slabs.

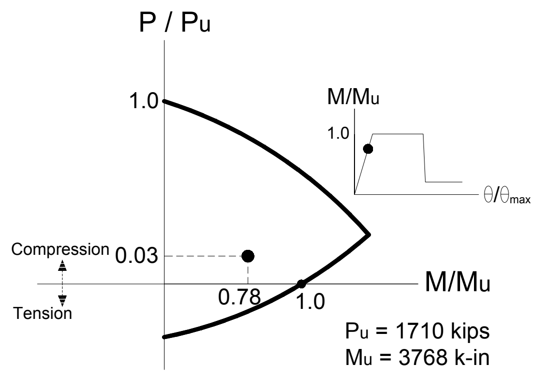


Figure 7.10 Axial force-moment relationship at location 1

Beams at the corner regions exhibited significant damage due to insufficient anchorage for the positive moment (dots in Fig 7.9). The cracking moment was reached and beams at these locations followed behavior shown in Fig 7.5. In Fig 7.11, the change in moment configuration from the negative bending to positive bending after a corner column removal is shown.

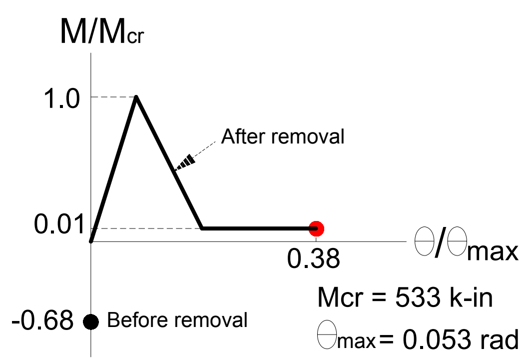


Figure 7.11 Insufficient positive moment capacity at corner regions

Although large rotations at the corner beams are observed, corner regions do not exceed rotational limits.

Shear behavior

Removal of a corner column did not cause shear failure at the mid-span in a beam member (location 2 in Fig 7.9). Shear demand at a beam mid-span was less than half of the available capacity as shown in Fig 7.12.

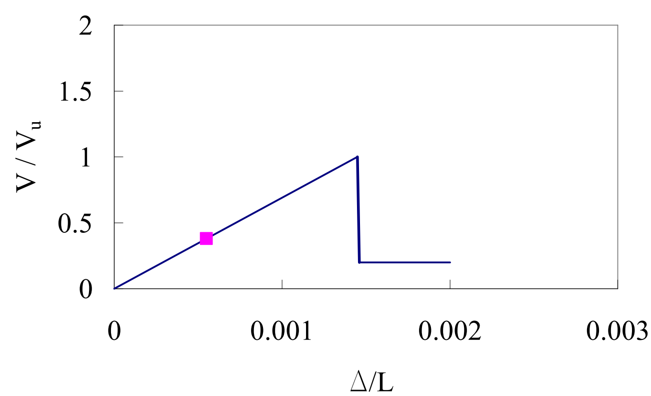


Figure 7.12 Shear force demand after removal of a corner column

Tension behavior in columns

Removal of a corner column did not cause lap splice failure in the column above the removed column (location 3 in Fig 7.9). After a corner column is removed, axial force demand changes from compression to tension. However, tension demand as shown in Fig 7.13 is small because the corner panel is successfully held by sufficient beam and slab capacities. The tension force capacity at an exterior column is reduced by 60% of the tension capacity due to consideration of insufficient lap splice length.

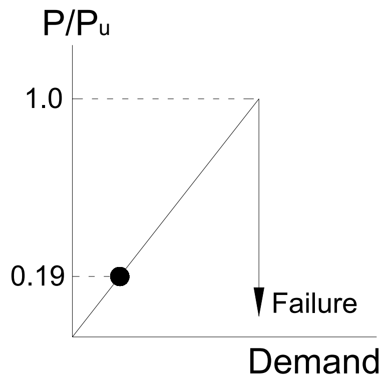


Figure 7.13 Axial force demand at a column above the removed column

7.2.1.2 Edge column

Flexure of beams and columns, shear in beams, and tension in columns are discussed when an edge column is removed.

Flexure behavior

When an edge column was removed, plastic hinges at several locations were developed (Fig 7.14). Frames [(a), (b), and (c)] are shown to indicate where plastic hinges form.

In frame (a), plastic hinges were not developed, although nominal capacity of a beam section was exceeded because the axial compression force at that section enhanced the beam capacity. Therefore, interior beams (location 1) at the perimeter edge that are located at the far end of the beam from the removed column location remained in the elastic range. Fig 7.15 illustrates axial force-moment interaction at location 1 in Fig 7.14.

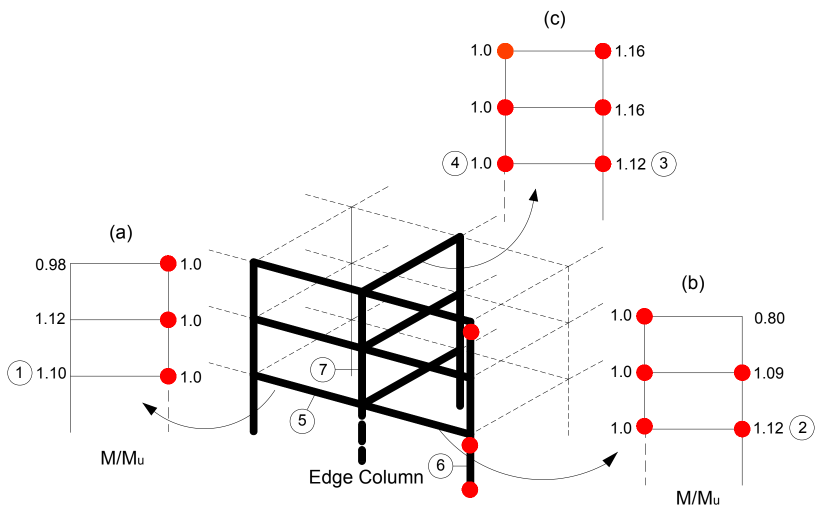


Figure 7.14 Locations investigated when an edge column is removed

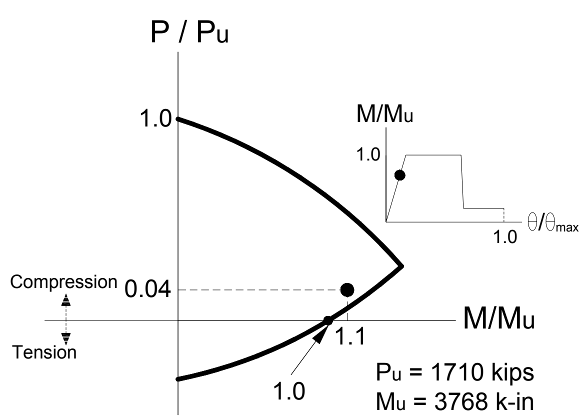


Figure 7.15 Axial force-moment interaction at location 1

Frame (b) shows plastic hinges formed at the end of beams at the second (location 2) and third floor after removal of an interior column because smaller top reinforcement (4-#7bars) was present than the interior region (6-#7bars). Axial force-moment interaction relationship at location 3 is exhibited in Fig 7.16.

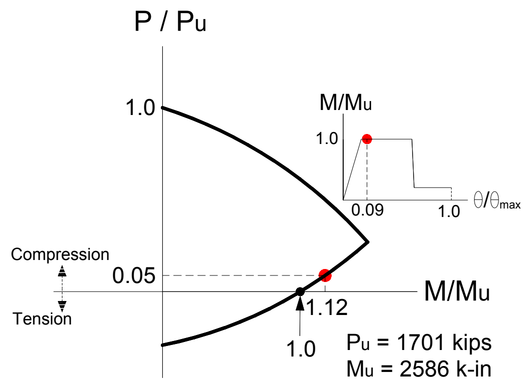


Figure 7.16 Axial force-moment interaction at location 2

After reaching the maximum moment capacity, the strength is maintained and the beam end rotation ratio reaches 0.09 (Fig 7.16) indicating that there is sufficient rotational capacity. Frame (c) exhibits plastic hinges at all beams, and the axial force-moment relationship at location 3 is shown in Fig 7.17.

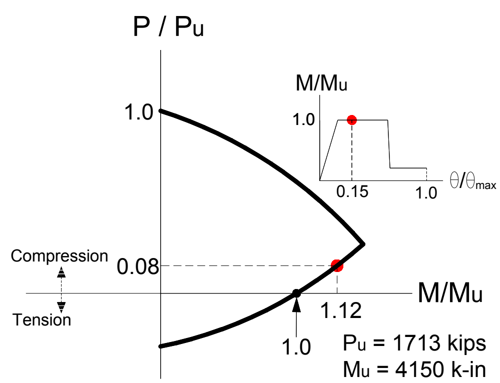


Figure 7.17 Axial force-moment interaction at second floor (location 3)

Although Frame (c) had the largest moment capacity, that moment capacity was reached because beams near the removed location reached their cracking moment (location 4) very quickly as shown in Fig 7.18.

Due to inadequate development length, all beams above the column that was removed reached their cracking moment as shown in Fig 7.14.

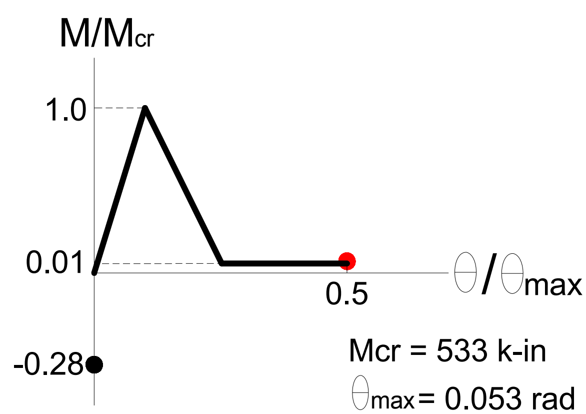


Figure 7.18 Change of moment configuration (location 4)

Corner columns (location 6 in Fig 7.14) exhibit plastic hinges at the first floor and third floor. Columns at the corner region experience much more moment demand compared with columns at the interior because corner columns are not restrained by adjacent members (interconnecting beams and slabs) as are those at the interior region. This restraint is likely the reason why a hinge at the first floor of the corner column was produced. Analysis results indicate such movement occurred.

Shear behavior

Shear demand location 5 in Fig 7.14 after removal of the edge column was below the shear capacity, and shear failure was not expected (Fig 7.19).

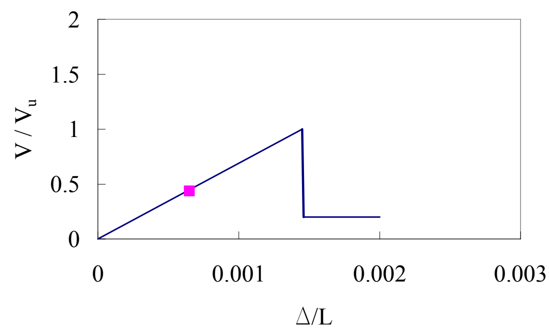


Figure 7.19 Shear force demand after removal of an edge column

Tension behavior in column

The tension force demand at the column above the removed column at location 7 in Fig 7.14 was well below the tension capacity based on actual lap splice lengths. Normalized axial demand to capacity ratio is given in Fig 7.20.

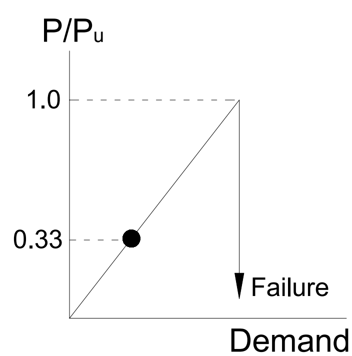


Figure 7.20 Axial force demand at a column above the removed column

7.2.1.3 Interior column

The plastic hinges that developed when an interior column was removed are shown in Fig 7.21.

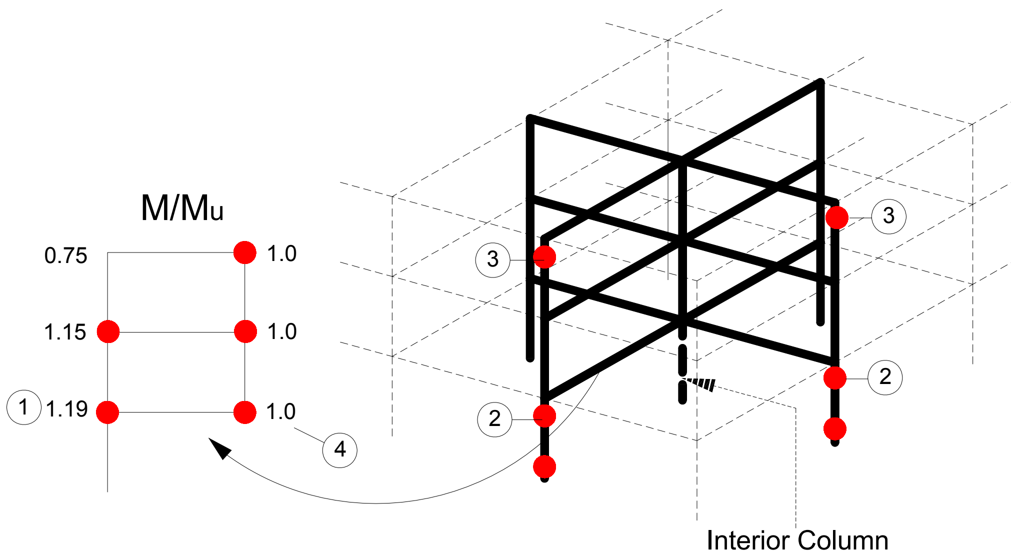


Figure 7.21 Plastic hinges formed when an interior column is removed

Flexure behavior

Plastic hinges at beam ends formed at the second floor and third floor. Due to insufficient development length of the bottom reinforcement, the cracking moment was reached in all beam ends at each floor above the column-removed location. The axial force-moment relationship of a beam hinge (location 1 in Fig 7.21) at the second floor is exhibited in Fig 7.22.

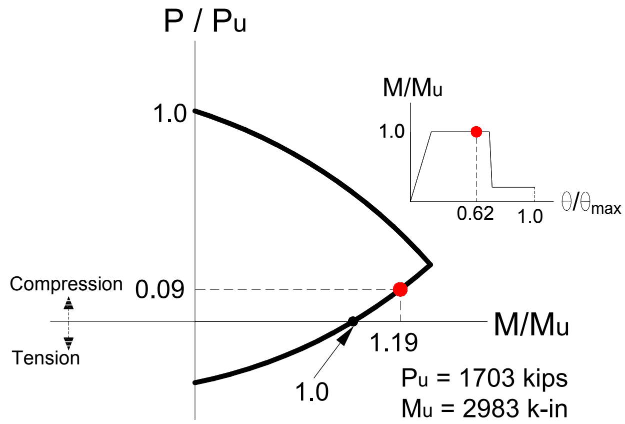


Figure 7.22 Axial force-moment interaction at location 1 in Fig 7.21

A moment larger than the nominal moment capacity was observed because compression forces from slab restraint resulted in enhancement of beam capacity. Compared with removal of a corner and edge column, very large rotations at beam ends were obtained because large tributary (effective) areas were loaded.

Column plastic hinges are shown in Fig 7.23(a) and (b) for location 2 and location 3, respectively. The column at location 2 had more axial force on it than that at location 3. Therefore, a larger capacity was developed [1.55 in Fig 7.23(a)]. On the other hand, the column at location 3 showed a very large end rotation because the deflection at the interior panel region was so large that columns at the exterior perimeter rotated much more than interior columns. Plastic rotation at location 3, as shown in Fig 7.23 (b), was very close to the rotational limit for that column. There is higher probability of exterior column failure.

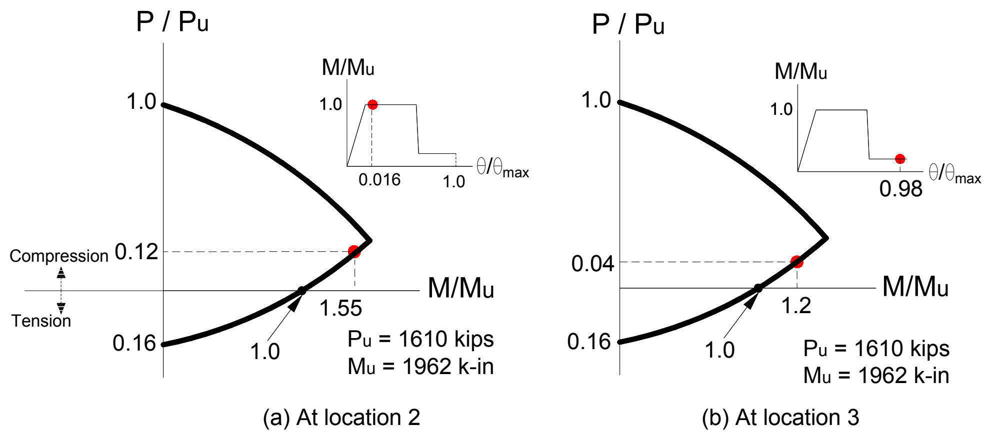


Figure 7.23 Column interaction diagram of hinges at the first and third floor

When an interior column is removed, beams bridging over the removed location have twice as large a span length as they originally had and cause moment to change at the removed location from negative to positive bending. Before the column is removed, a large negative moment ratio (-2.34 times the cracking moment) is present at the beam end (location 4).

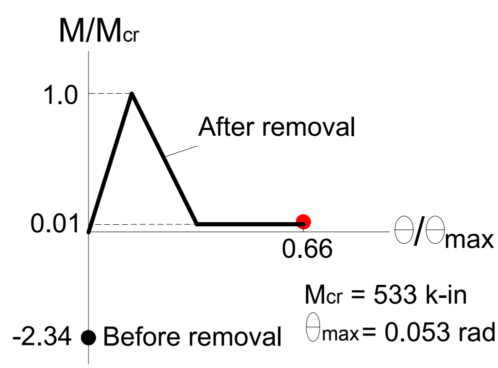


Figure 7.24 Cracking moment reached at location 4

Adjacent slabs also played important roles in supporting large tributary areas to develop alternate load paths. The maximum slab rotation reached approximately 25% of the slab rotational capacity.

Shear behavior

Shear demand at mid-span of a beam did not reach shear capacity when an interior column was removed. Shear failure at mid-span did not occur as shown in Fig 7.25.

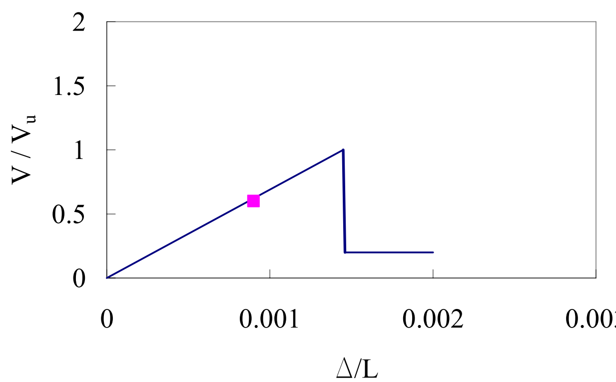


Figure 7.25 Shear force demand after removal of an edge column

Tension in columns

Lap splice failure in a column was not observed at the design load (144 psf for floor) when an interior column was removed. Larger tension strength at interior columns was considered because longitudinal reinforcement at interior columns were larger. Although removing an interior column caused the largest tension force on the column above the removed location, the tension demand did not exceed its capacity as shown in Fig 7.26.

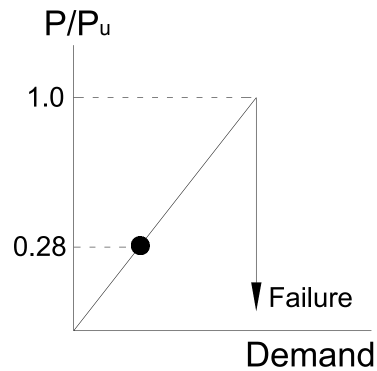


Figure 7.26 Axial force demand at a column above the removed column

7.2.2 Comparison of deficient and robust structures

The cost of changing a deficient structure to a robust structure is quite small if details are changed during initial design. Correction of deficiencies in existing structures will be expensive. Structural performance can be improved by providing structural integrity reinforcement, close spacing of transverse reinforcement in a beam, and sufficient splice length in a column. Assuming a deficient structure is retrofitted to make a robust structure, analytical results between deficient and robust structures are compared for removal of an edge column. When an edge column is removed, nonlinear response of structures under the same gravity load (144 psf) is compared in Fig 7.27.

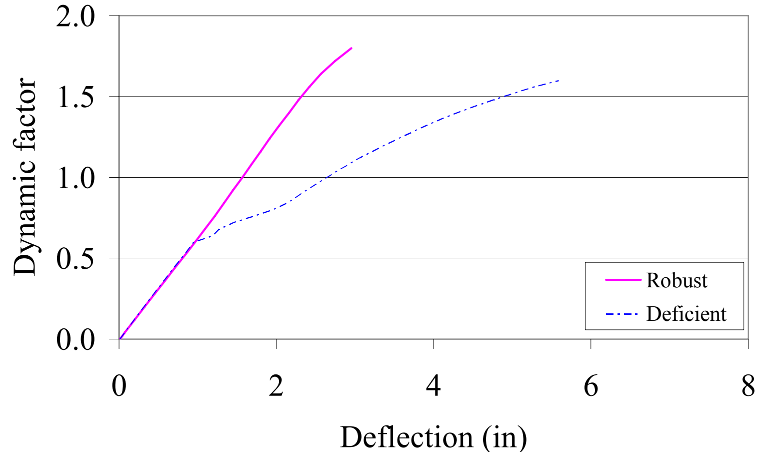


Figure 7.27 Comparison between robustness and deficiency

Under the service load (144 psf), energy balance in a robust structure is realized at a deflection of about 3 in. The deficient structure reaches balance at 5.6 in. In addition, most regions of a robust structure remain elastic. This indicates that a robust structure has reserve capacity to absorb more energy than a deficient structure.

Moment-rotation behavior at a beam close to the removed location for robust and deficient structures is compared when an edge column is removed. Robustness and deficiency in terms of structural integrity are plotted in Fig 7.28, and moment-rotation capacities for two different cases are compared in Fig 7.29.

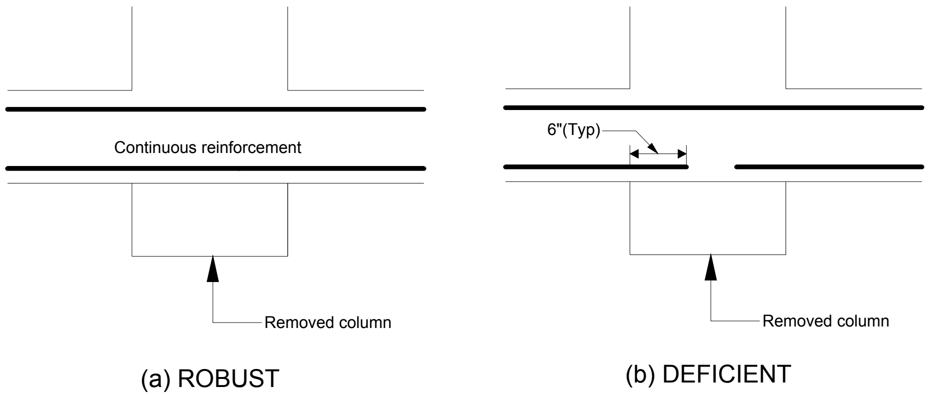


Figure 7.28 Comparison of reinforcement details

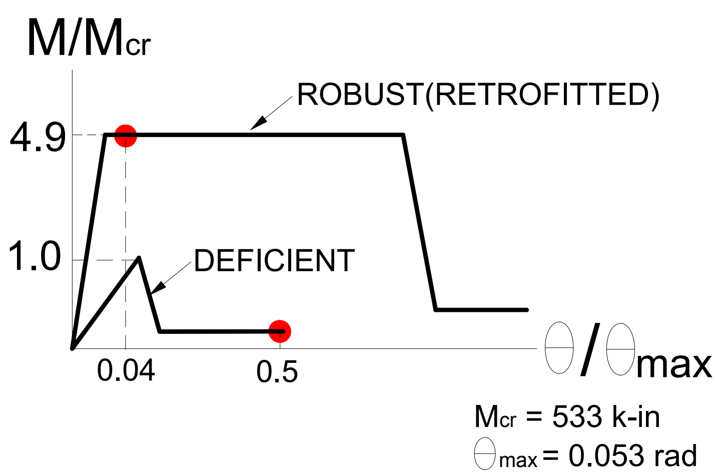


Figure 7.29 Capacity comparison between the robust and deficient structures

Continuity of the bottom reinforcement at a connection region makes a huge difference in a load-carrying capacity and ductility, as shown in Fig 7.29.

Alternate load paths or specific local resistance can be used to improve structural performance. Alternate load paths can be provided to limit the effects of damage. The analytical results indicate that deficient structures are able to carry dynamic load effects through alternate load paths that take advantage of the ductility in adjacent supporting components. The development of alternate load paths is most easily accomplished in new construction of a monolithic structure. It is likely to be more difficult with precast concrete structures.

Although it is very difficult to determine magnitude and location of specific threats, specific local resistance by wrapping structural components with fiber composites are often used. However, wrapping a column with CFRP does not eliminate the possibility of column failure against external threats because there are always possibilities for external threats to exceed strengthened column capacities.

7.3 CONCRETE STRUCTURES WITHOUT INTERIOR BEAMS

A robust beam-column RC structure exhibited excellent resistance to abnormal loads, thereby limiting the potential for progressive collapse. The performance of beam-column frames is enhanced through the interactions of beams, slabs, and columns. Ductility provided by the various elements helps a structure reach good performance.

In structures without interior beams, redistribution of forces may be more limited than those with interior beams. When an extreme event occurs, these structures may be at great risk for progressive collapse.

A flat plate concrete structure was modeled. The flat plate model structure had span lengths of 20 ft (both directions) and story heights (12 ft). The modeled structure is similar to a flat plate structure of the PCA document [35], originally designed to resist seismic and wind forces. The modeled structure had sufficient strength to meet load-combinations. In order to simulate a deficient structure, insufficient development length at connection regions, inadequate transverse reinforcement at beams, and short column lap splices were assumed.

7.3.1 A deficient flat plate structure

The model for a deficient flat plate structure is shown in Figs 7.30 and 7.31. Material properties used in modeling are illustrated in Table 7.4. Concrete and steel have the same properties shown in Figs 5.11 and 5.12. A strength increase factor of 1.25 is used for concrete and steel. Spandrel beams of 18 in by 18 in are framed at the exterior perimeter. All columns are 18 in by 18 in with 8-#9 bars tied by 9 in spacing. A typical slab thickness of 8 in is used.

The applied gravity load is summarized in Table 7.5. Dynamic effects were examined by obtaining a factor of α.

Reinforcement details for beams and slabs taken from the PCA document [35] are shown in Figs 7.32 and 7.33. Beam moment capacities at selected locations in Fig 7.34 are summarized in Table 7.8.

Table 7.4 Material properties

Concrete	4 ksi
Steel	60 ksi

Table 7.5 Applied gravity load [35]

Dead load	100 psf (Floor) + 30 psf (Superimposed) = 130 psf Weight of beams, columns, and partitions = 14 psf
Live load	50 psf (Floor), 30 psf (Roof)
Total load	Dead load + 0.25 Live load 157 psf (Floor), 151 psf (Roof)
Applied Load	α (Dead load + 0.25 Live load) for static analysis (Dead load + 0.25 Live load) for dynamic analysis

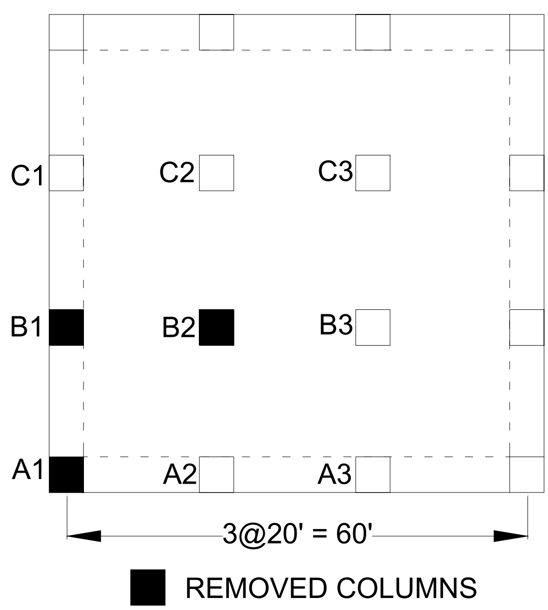


Figure 7.30 Plan view of a flat plate structure

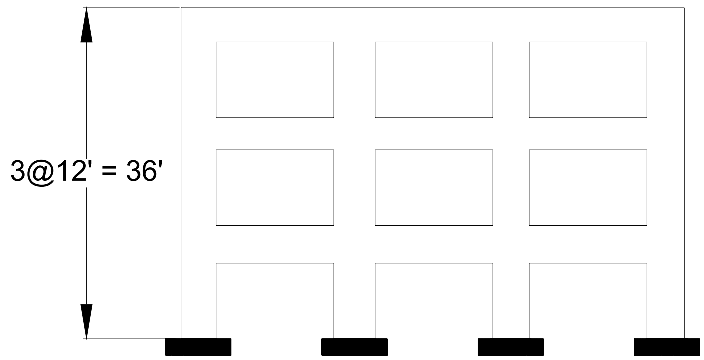
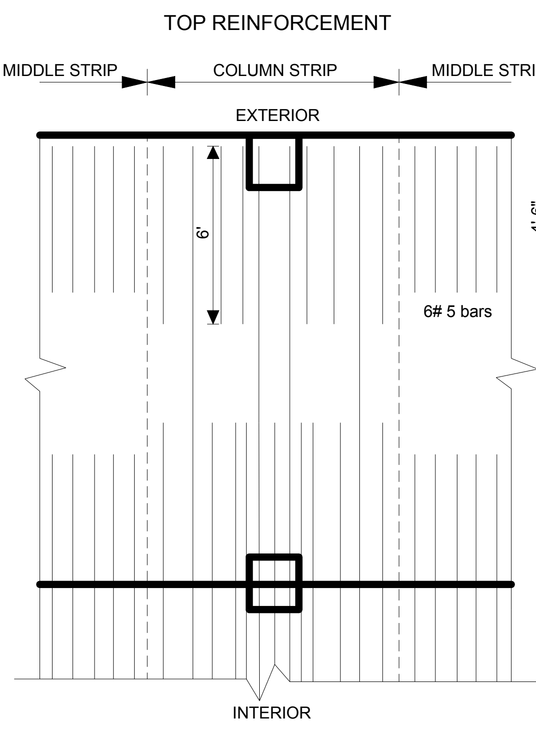
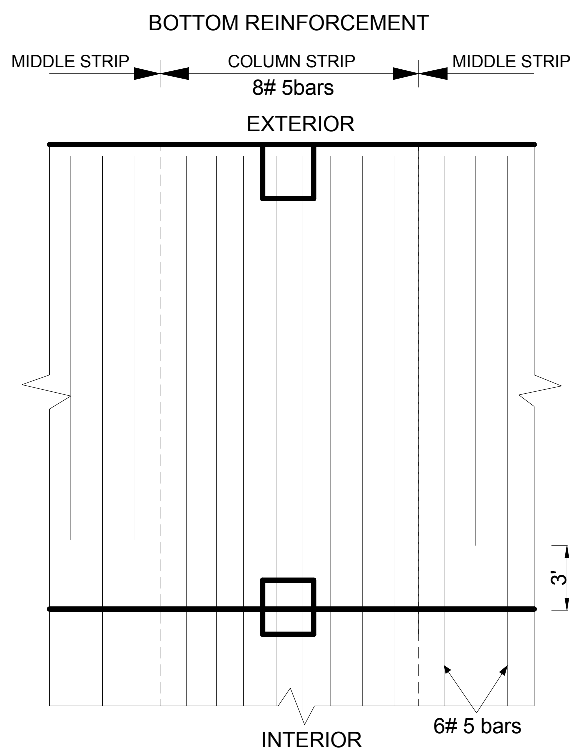


Figure 7.31 Elevation view of a flat plate structure



(a) Top slab reinforcement



(b) Bottom slab reinforcement

Figure 7.32 Reinforcement details in slab [35]

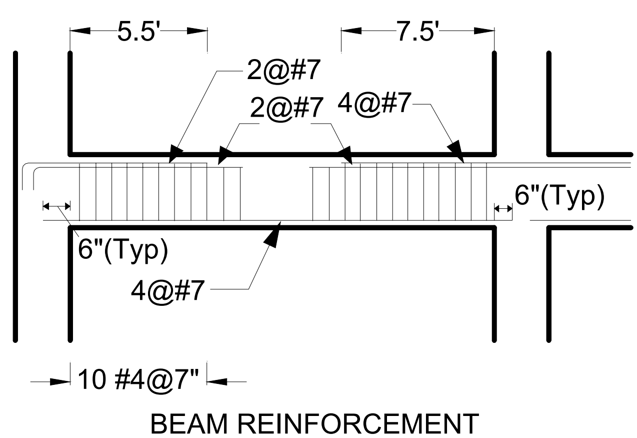


Figure 7.33 Reinforcement details in beams [35]

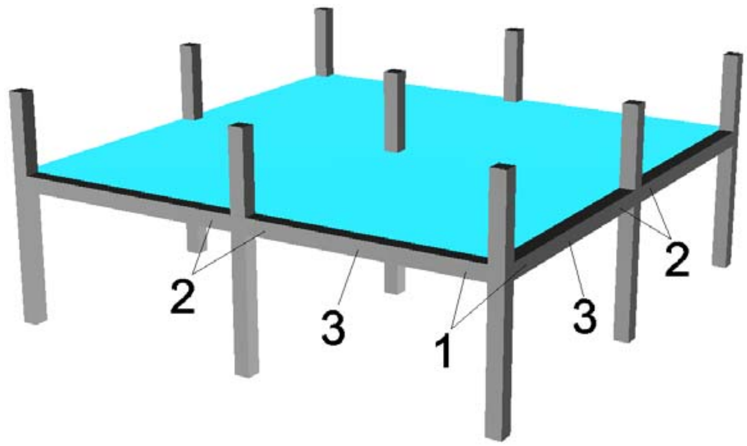


Figure 7.34 Beam elements of the flat plate structure

Table 7.6 Positive and negative moment capacities for beams

Location	Positive moment (k-in)	Negative moment (k-in)
1	600	2870
2	600	4030
3	2600	600

Insufficient development length of the bottom reinforcement at connection regions was considered based on Fig 7.3 and was applied in modeling using Fig 7.5. Inadequate transverse reinforcement in beams for shear was also considered based on Fig 7.6. Insufficient column lap splice for tension was applied in modeling using Fig 7.7. Large deflections in elements were considered for each analysis.

7.3.1.1 Nonlinear static and dynamic analyses

Columns shown in Fig 7.30 were removed to investigate dynamic factors and deflections. Applied loads were 157 psf on the floor and 151 psf on the roof. Dynamic factors and deflections for each case are summarized in Table 7.7 and plotted in Fig 7.35. Deflection was observed at the location above a removed column.

Table 7.7 Deflection and dynamic factor for each column removed (157 psf)

Removed location	Nonlinear Analysis	Load	Deflection (in)
Corner Column	Static*	***1.76*** x 157 psf	*2.33*
	Dynamic	157 psf	*2.15*
Edge Column	Static*	***1.75*** x 157 psf	*3.83*
	Dynamic	157 psf	*3.80*
Interior Column	Static*	***1.68*** x 157 psf	*11.7*
	Dynamic	157 psf	*11.7*

* With dynamic factor

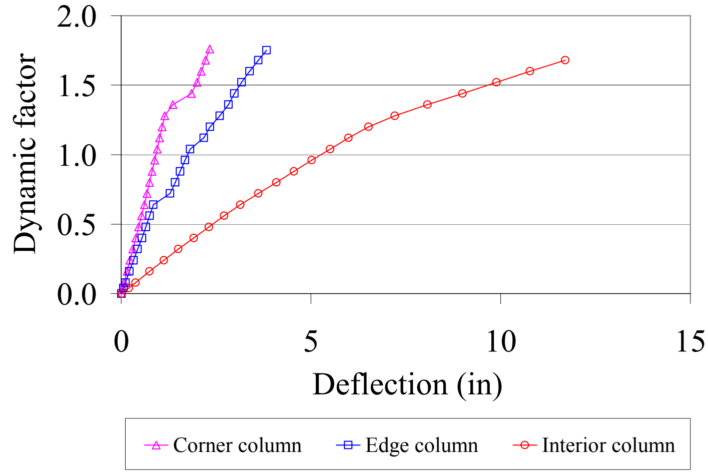
The flat plate structure adequately resisted dynamic load effects when each column was removed. Balance between external work and internal energy was reached for removal of each column.

Dynamic factors ranged from 1.68-1.76. Deflections from static analysis including dynamic factors match with those from dynamic analysis. Removal of a corner and edge column shows small differences in deflection between nonlinear static and dynamic analyses because plastic hinge formation may be somewhat different between two analyses and distributed masses for dynamic analysis may generate this difference.

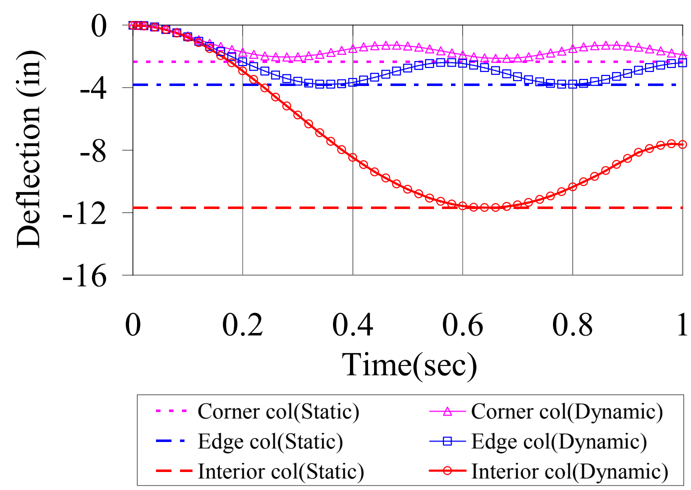
Removal of a corner column resulted in the lowest deflection (approximately 2 in) and a dynamic factor of 1.76. The structure did not collapse because adjacent slabs and exterior beams were able to carry additional loads when the dynamic effects of corner column removal were considered. Alternate load paths were utilized well.

When an edge column was removed, a dynamic factor of 1.75 was calculated, and a deflection of approximately 4 in was observed at the removed location. Larger effective areas than the removed case of a corner column were observed. Adjacent exterior beams and slabs may have helped the floor to resist more dynamic effects, allowing greater deflections than the corner column case.

Removal of an interior column generated very large deflections because the load area (tributary area) was very high. A dynamic factor of 1.68 and deflection of 5% of the span length (11.7 in) were obtained. Much greater gravity loads were applied on the effective areas. This caused a larger deflection than when corner and edge columns were removed.



(a) Nonlinear static analysis



(b) Nonlinear dynamic analysis

Figure 7.35 Dynamic factors and deflections for nonlinear analyses

Abrupt increase in deflection [Fig 7.35 (a)] for nonlinear static analysis (kinks) represents the redistribution when the cracking moment is reached and the loss of strength at beams above the removed locations occurs.

Each removal case showed variation in natural periods. In Fig 7.35, as a structure absorbs more inelastic energy (deflections increase), the natural period increases. Shorter natural periods were observed when most of the flat plate structure remained in the elastic region after a corner column was removed.

7.3.1.2 Punching Shear Failure

The shear capacity at the critical perimeter (25 in by 25 in) shown in Fig 7.36 was compared with shear demand.

Removal of a corner column (A1 in Fig 7.30) was not considered because spandrel beams provided sufficient resistance. Removal of edge (B1) and interior (B2) columns was investigated. When an edge column (B1) was removed, the possibility of punching shear failure at B2 location was examined. When an interior column (B2) was removed, column perimeters at C2 or B3 were considered as critical locations for punching shear failure.

Two-way shear capacity using ACI 318 [1] at the critical perimeter was 198 kips (282 psi). There was no transverse reinforcement in the slabs. The one-way shear capacity was also checked but did not control failure. Two-way shear capacity was compared with the shear demand resulting from the load across the critical perimeter after removal of a column. Longitudinal reinforcement passing through the critical perimeter is shown in Fig 7.37.

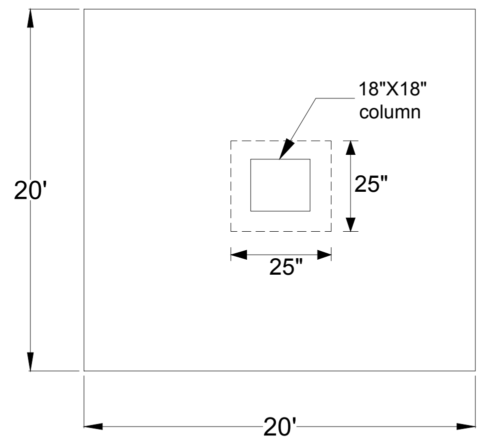


Figure 7.36 Critical perimeter for two-way shear

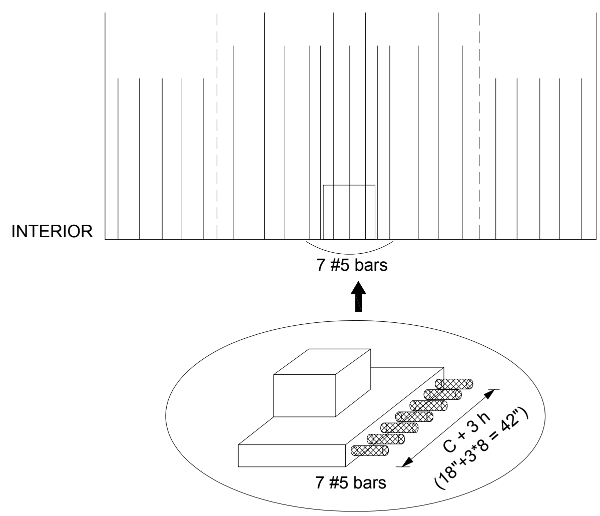
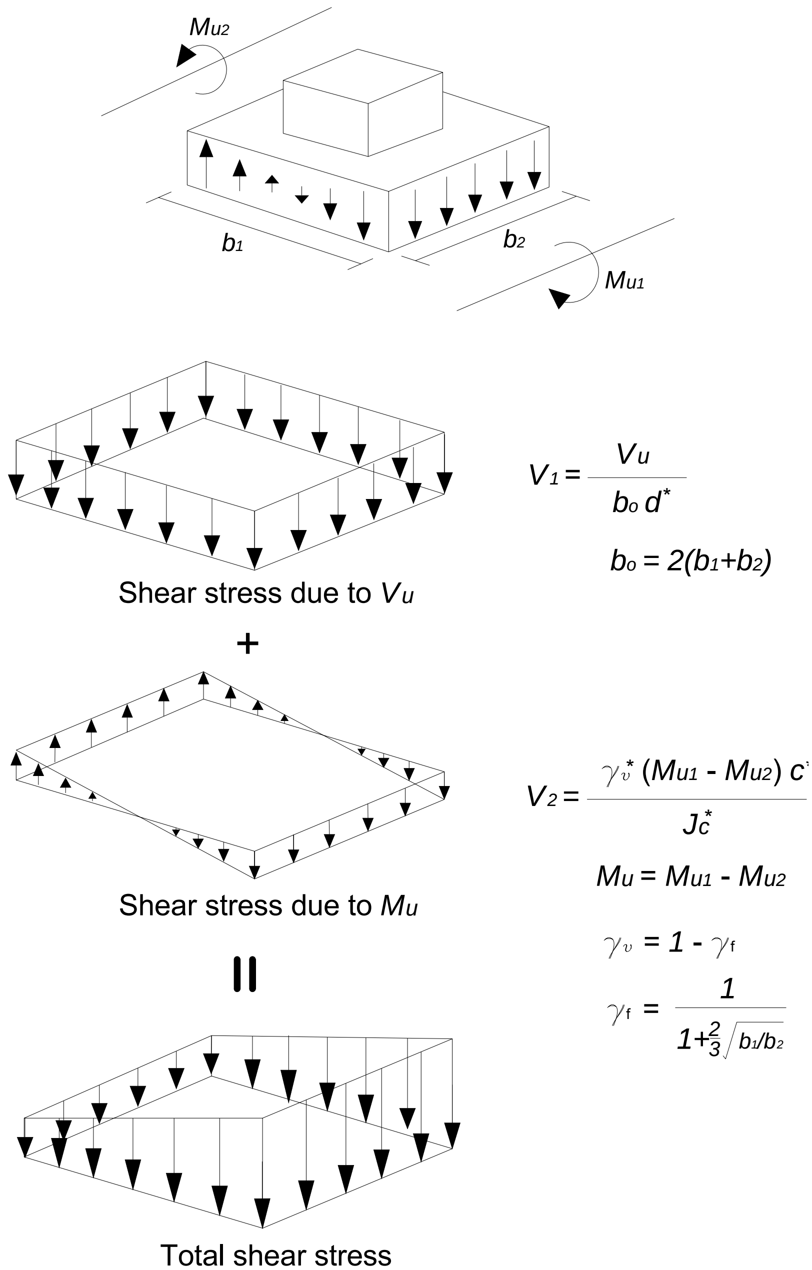


Figure 7.37 Reinforcement passing through a critical section

Shear demand at the critical sections include shear (V_u) caused by the applied gravity load plus shear caused by unbalanced moment (M_u) at the shear perimeter. Figure 7.38 illustrates shear forces acting on the critical section.



* γ_v: Fraction of the unbalanced moment transferred by shear
c : Distance from centroid axis to shear perimeter
d : Distance from top fiber to the reinforcement in a slab
J_c : Polar moment of inertia of the shear perimeter about the centroid axis

Figure 7.38 Shear stress considered in a shear perimeter [45]

As shown in Fig 7.38, if the total shear stress (demand) exceeds the shear capacity, punching shear can be expected. Shear stress (V_1 in Fig 7.38) resulting from the applied gravity load was calculated from the static analysis for the given load shown in Table 7.8. Unbalanced moment (M_u) was obtained at the shear perimeter from the analytical results. This unbalanced moment is transferred to the column by flexure in the slab over an effective width (the width of the column plus 3 times the slab thickness as shown in Fig 7.37), which produces shear stress (V_2 in Fig 7.38) on the critical region [1]. The fraction of unbalanced moment transferred through shear is given as γ_v, and the fraction of unbalanced moment transferred through flexure is given as γ_f. For interior column, the value of γ_f can be increased by up to 25%, provided that V_u does not exceed $0.4\phi V_c$ [1].

The unbalanced moment for removal of each edge and interior column are shown in Figs 7.39 and 7.40.

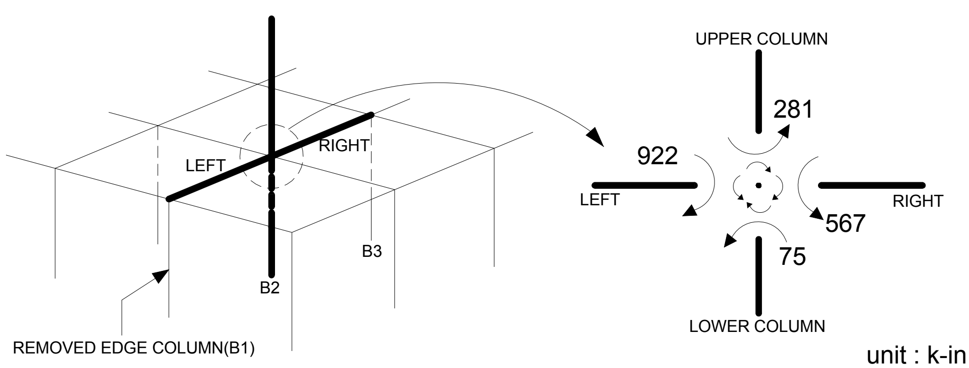


Figure 7.39 Unbalanced moments at the adjacent column (edge column removal)

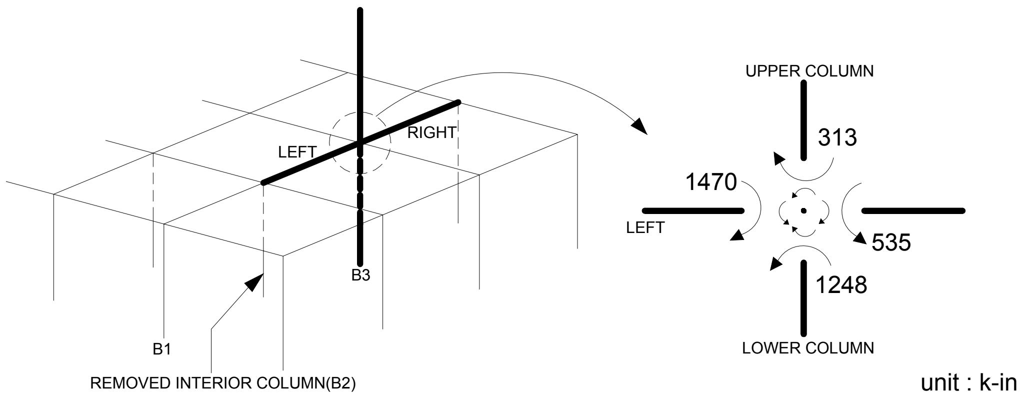


Figure 7.40 Unbalanced moments at the adjacent column (interior col removal)

Shear demand (V_u) around the critical perimeter was compared with factored shear capacity (ϕV_c), where $\phi = 0.75$ and $V_c = 4\sqrt{f_c^{'}} b_o d$. In both cases, values of $V_u / \phi V_c$ were larger than 0.4 resulting in some shear stress due to unbalanced moment at the shear perimeter, as shown in Table 7.8. Calculation example at location B3 is given in Table 7.9. Shear stress (*V_1)* by the gravity load and Shear stress (*V_2)* resulting from the unbalanced moment are calculated based on equations given in Fig 7.38.

Table 7.8 Shear demand at the perimeter

Removed column	Locations	$\frac{V_u}{\phi V_c}$	Shear by applied load* (V_1, psi)	Shear by the unbalanced moment * (V_2, psi)	Total shear stress (psi)
Edge column(B1)	B2	0.73(>0.4)	156	24	180
Interior column(B2)	B3 (First floor)	0.71(>0.4)	150	63	213

* Include dynamic factor

Table 7.9 Calculation examples for the shear stress

Location	B3 (First floor)
Modification for γ_f	$\frac{V_u}{\phi V_c} = \frac{105\ kips}{0.75 \cdot 198\ kips} = 0.71 \rightarrow \gamma_f = 0.6,\ \gamma_v = 0.4$
V_1 (psi)	$V_1 = \frac{V_u}{b_o d} = \frac{105000\ lbs}{100\ in \cdot 7\ in} = 150\ psi$
V_2 (psi)	$V_2 = \frac{\gamma_v (M_{u1} - M_{u2}) c}{J_c} = \frac{0.4 \cdot (935\ k \cdot in) \cdot 25\ in/2}{74346\ in^4} = 63\ psi$ $where\ M_{u1} - M_{u2} = (1470 - 535) = 935\ k \cdot in$
$V_1 + V_2$ (psi)	*213 psi*

Shear capacity (282 psi) at the perimeter of the adjacent column locations after removal of a column was not exceeded. Because the greatest risk of punching shear failure exists at the first floor for a column removed below that floor, punching shear failure at the second and third floor is not expected.

7.3.1.3 Detailed structural behavior

- Corner column removed

Most regions at a corner panel exhibit elastic response as shown in Fig 7.41 except beam ends at the corner region. This region (location 2 in Fig 7.41) reaches cracking for the positive moment due to inadequate embedment length. The axial force-moment interaction at a beam end (location 1) is shown in Fig 7.42, and it indicates sufficient capacity for flexure.

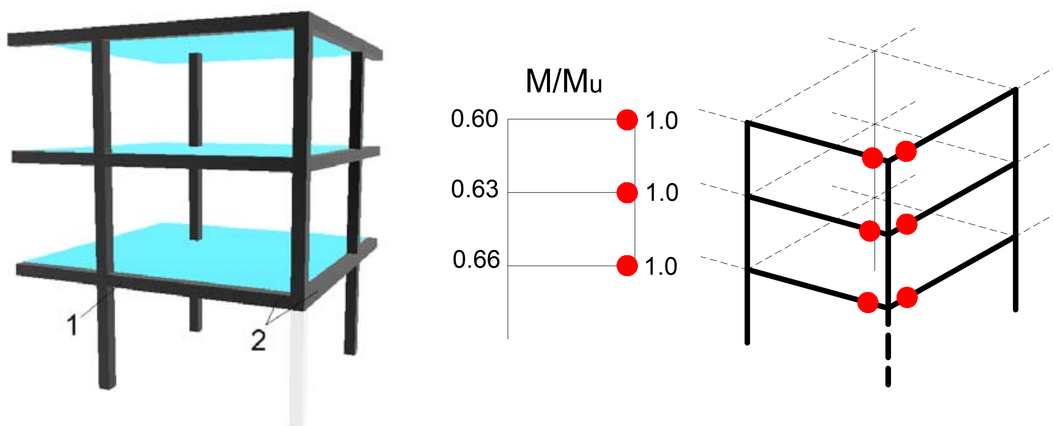


Figure 7.41 Plastic hinges at observed locations

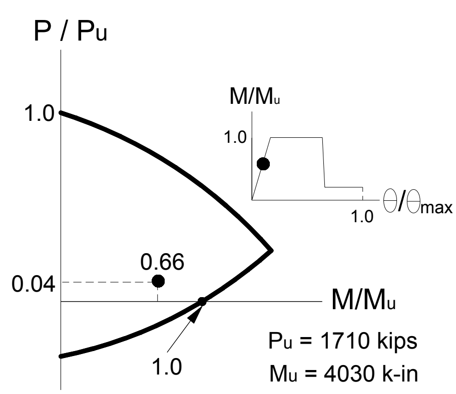


Figure 7.42 Axial-moment interaction at location 1

Shear demand at the middle of the beam is 35% of the concrete shear capacity. The tension force in a column above the removed corner column is 8% of the column capacity in tension even though insufficient lap splice lengths are considered. Shear failure and lap splice failure do not occur when a corner column is removed.

Although beams at the corner region are damaged, alternate load paths are well established. Therefore, progressive collapse is not expected for this case.

- Edge column removed

Plastic hinges formed at beam ends as shown in Fig 7.43. Due to insufficient development length of the bottom reinforcement at connection regions, the cracking moment (number of 1) is reached at all beam ends at the removed column location 1 as shown in Fig 7.43. Residual strength and plastic rotation at locations 1 are shown in Fig 7.44.

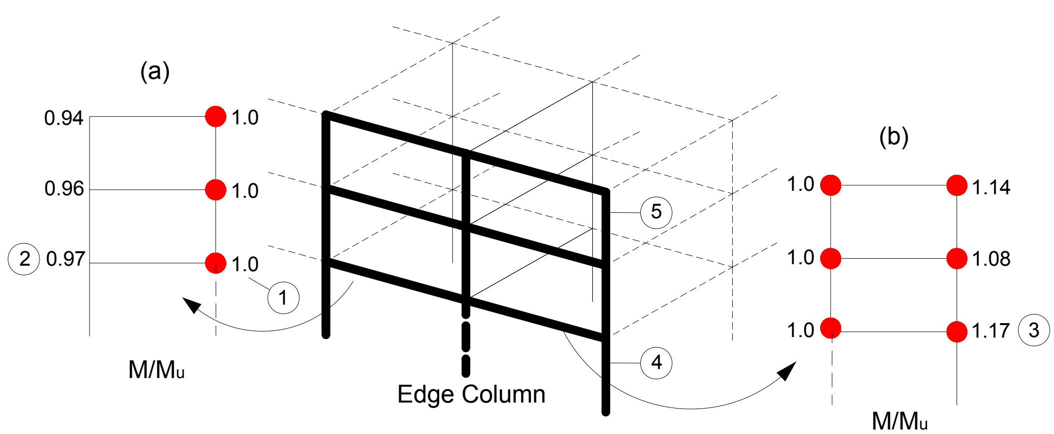


Figure 7.43 When an edge column is removed

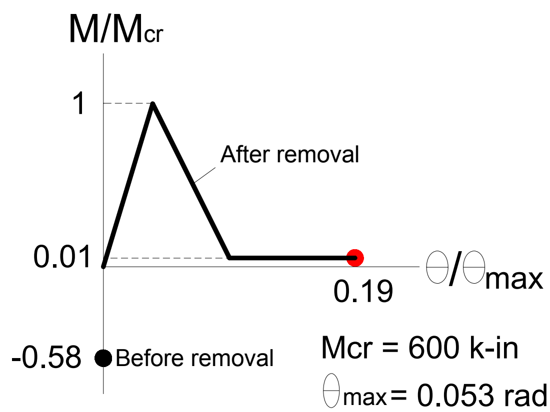


Figure 7.44 Plastic hinge behavior at location 1

The bending moment reverses from negative bending to positive bending after an edge column is removed. Plastic rotations at the beam ends are less than the allowable rotation regulated by the UFC rotation limit (0.053 radian). Moment redistribution is well developed. Figure 7.45 illustrates the axial force-moment relationship of beam ends at locations 2 and 3.

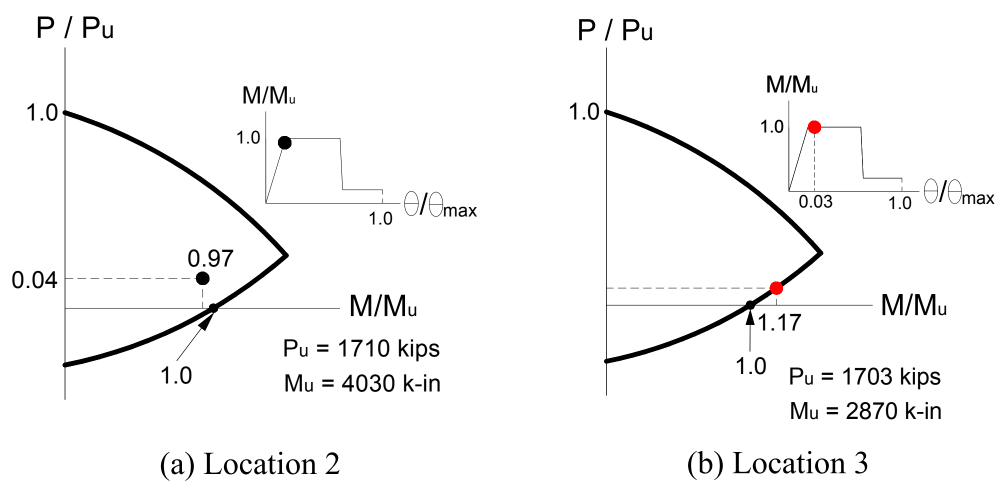
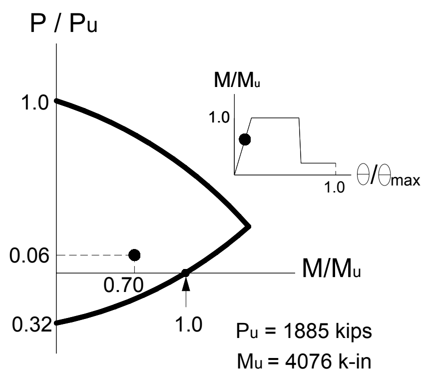
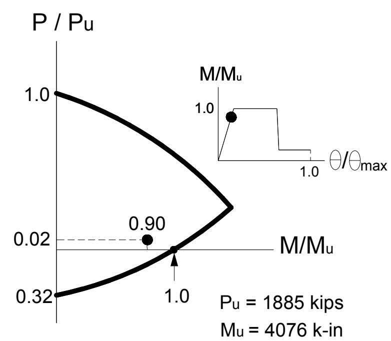


Figure 7.45 Axial force-moment interaction at beam ends

Columns at locations 4 and 5 are also observed for possibility of failure. Column interaction diagrams for these two columns are provided in Fig 7.46. The column at location 5 carried more moment and less axial force than the column at location 4 Therefore, the top column can be considered more vulnerable to failure. The GSA sidesway limit for this column was not exceeded.





(a) Location 4 (b) Location 5

Figure 7.46 Axial force-moment interaction at a corner region

The shear force at the mid-span of beams reached approximately 50% of the concrete shear strength and capacity was not reached. Shear failure at mid-spans were not observed.

Axial tension for the column above the removed edge column reached 15% of the tension capacity of that column considering insufficient lap splice length. Therefore, lap splice failure for tension in that column is not expected.

- Interior column removed

When an interior column is removed, columns at the exterior perimeter are investigated. The effective stiffness of exterior columns to lateral bending is different from the interior columns. Exterior columns frame into the spandrel beams around the exterior perimeter and interior columns do not have beams. However, exterior columns are more vulnerable to failure because they are more easily damaged by terrorists or through vehicular impact and they lack in the continuity that is provided by surrounding slabs in comparison with interior columns. Fig 7.47 shows locations of columns investigated and column interaction diagrams are provided in Fig 7.48.

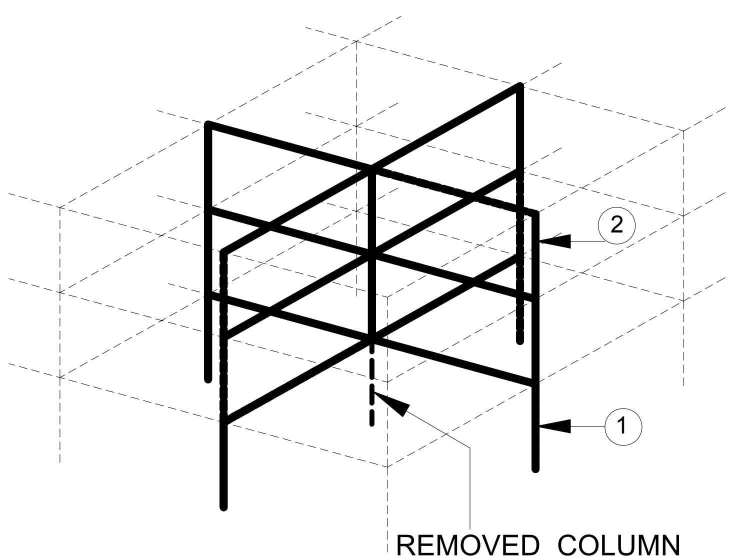
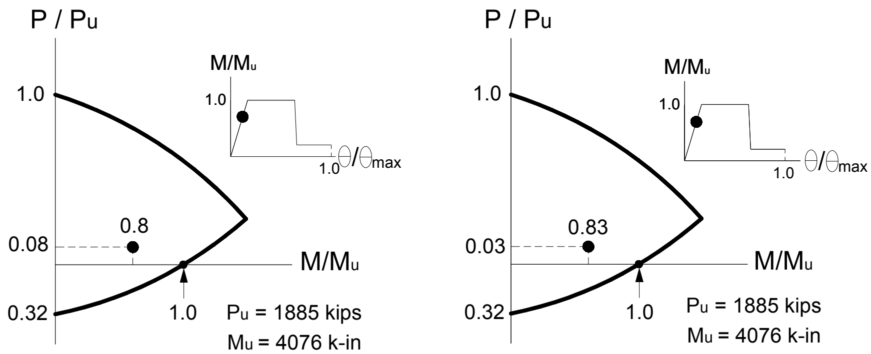


Figure 7.47 Columns investigated



(a) Location 1

(b) Location 2

Figure 7.48 Axial force-moment interaction at each column

As shown in Fig 7.48, the columns had sufficient capacity. After an interior column is removed, interior slabs are subjected to loads increased by dynamic effects. From the analytical results, slab reinforcement yields under positive moments at the removed location, and yielding is present at negative moment sections on the surrounding perimeter that passes through the adjacent columns. Red meshes in Fig 7.49 illustrate yielding in slab reinforcements.

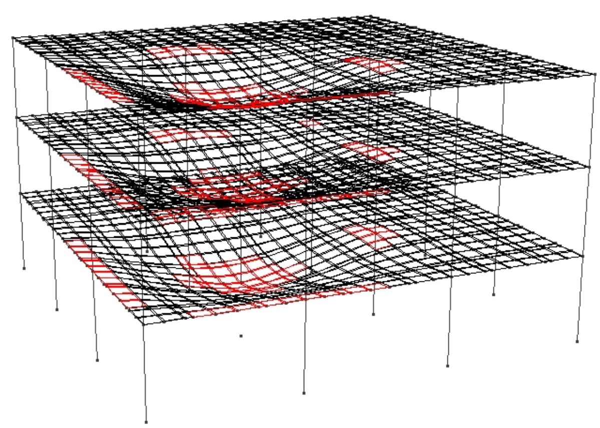


Figure 7.49 Yielding in slab reinforcements

Slab rotation is compared with the GSA and UFC criteria (6^o, 0.105 radian); 53% of the slab rotational limit was reached. Therefore, sufficient rotational capacity enables these slab regions to carry additional loads.

7.3.2 Modification of flat plate response

Analytical results indicate that interior areas in a flat plate structure are most likely to result in damage and threaten life safety due to progressive collapse. Concrete columns located at the exterior perimeters are most vulnerable to external threats such as collisions of vehicles or explosions. If alternate load paths are not provided, local collapse or global collapse disproportionate to local failure may occur. Therefore, rehabilitation in a flat plate structure should consider strengthening interior columns as well as exterior columns.

Although removal of corner and edge columns did not cause progressive collapse for the case considered, exterior columns are more prone to external attacks. If severe deficiencies exist, these deficiencies should be taken into account by rehabilitating the structures to provide alternate load paths. Detailing at a connection using the seismic resistance criteria recommended in the ACI code may provide alternate load paths. If it is very difficult to modify a structure, composite materials around the perimeter as shown in Fig 7.50 can improve structural performance when external loads are applied. Specific local resistance such as wrapping a column with composite materials or steel jackets as shown in Fig 7.50 can be used. Although wrapping a vertical element with composite materials is a common method to resist external loads, it is very difficult to predict the magnitude of an external attack. Therefore, specific local resistance can be considered as an option when alternate load paths are very difficult to provide.

Because flat plate structures with deficiencies did not develop alternate load paths when an interior column was damaged and lost load carrying capacity, retrofit schemes should focus on strengthening interior columns and providing alternate load paths when the interior column is severely damaged. Slabs containing high reinforcement ratios in the column strip region resisted abnormal loads well, and punching shear failure is not likely. Even if punching shear failure occurs, sufficient continuous reinforcement at the top and bottom of the slab through the connection regions may support the isolated slabs and prevent progressive collapse as shown in Fig 7.51. However, this continuous reinforcement can only be established for a newly constructed flat slab structure. For existing structures, composite materials can be used to provide continuity at the interior connection region as shown in Fig 7.52.

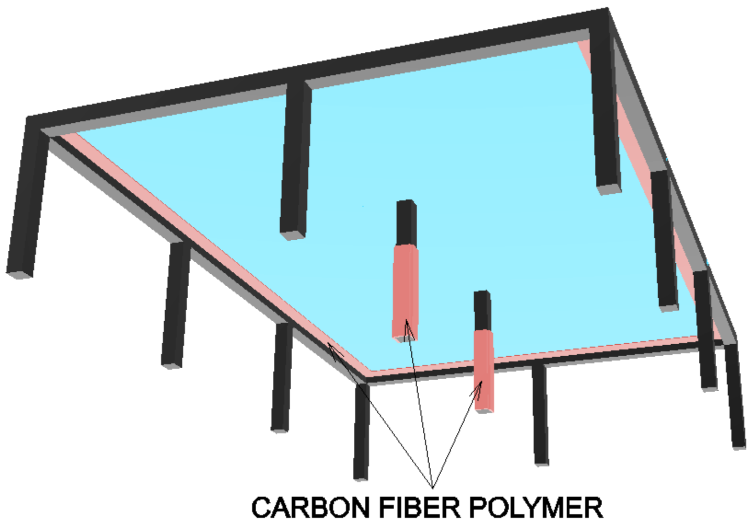


Figure 7.50 Retrofitted flat panel structure at the perimeter

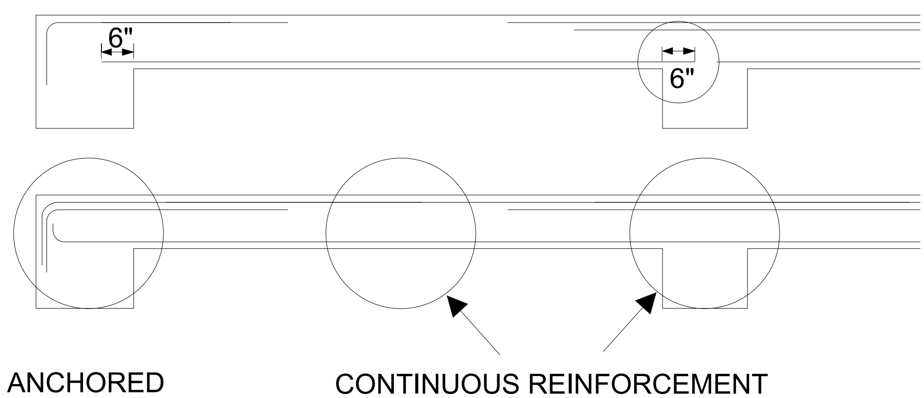


Figure 7.51 Continuous reinforcements in a slab

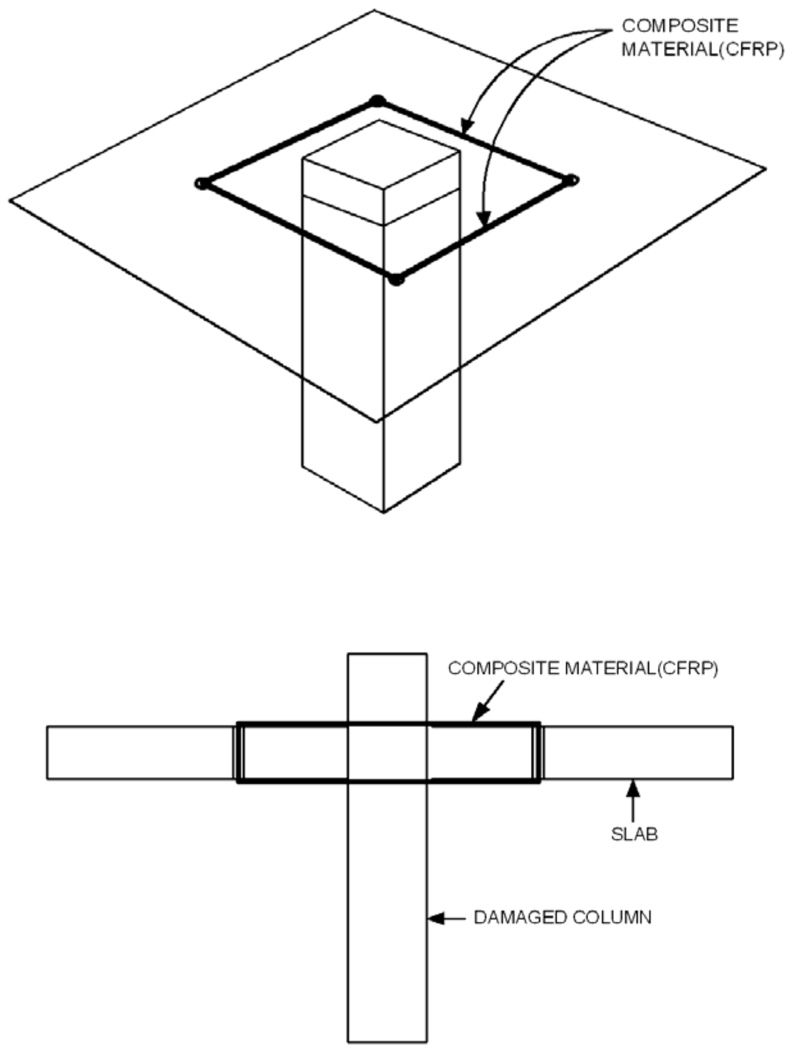


Figure 7.52 Use of composite materials for continuity

CHAPTER 8

Summary and Conclusions

8.1 SUMMARY

In this dissertation, the response of structures when a column is removed was studied. The GSA and UFC guidelines require notional removal of columns to simulate the effects of various abnormal loadings on structures. Linear static, linear dynamic, nonlinear static, and nonlinear dynamic analyses were conducted. Nonlinear static analyses were compared with results from experiments, and analytical results from nonlinear dynamic analysis were compared with those from research reported in the literature.

A robust concrete frame structure was investigated to verify dynamic effects suggested by the GSA guidelines. Deficient frame structures and flat plate structures were investigated to study the development of alternate load paths in structures to reduce the risk of progressive collapse.

The effect of deficiencies found in older concrete structures was demonstrated: insufficient development length of bottom reinforcement, inadequate transverse reinforcement for shear, and insufficient lap splice lengths in a column. Dynamic effects resulting from removal of a critical column were compared, and detailed moment-rotation behavior at hinges, shear failure in the middle of beams, and axial load capacity in columns were investigated. Analytical results comparing a robust structure and deficient structure were provided.

8.2 CONCLUSIONS

Concrete frame and flat plate structures were modeled and analyzed by using nonlinear static and dynamic analyses. Detailed structural responses were examined after a critical column was assumed to have lost its load-carrying capacity. Structural responses are observed as follows:

- **Dynamic Effect of Removal of a Column**

  Dynamic factor

  A robust concrete frame structure generated various dynamic factors (1.45-2.0) from nonlinear static analysis. Nonlinear dynamic analysis matched static analysis when the dynamic factor was used to amplify the applied loads. A robust structure showed sufficient resistance for the additional loads produced by the dynamic effects of removal of a critical column.

  A deficient structure showed smaller dynamic factors (1.52-1.72) when columns were removed. These dynamic factors are smaller than given in the GSA and UFC guidelines (2.0). Smaller dynamic factors for a robust and deficient structure were calculated based on consideration of dynamic effects generated by the gravity loads. There are possibilities of additional dynamic effects from the collapse of the upper floors and limited load-redistribution caused by significant damage near the connection regions above the removed column. Therefore, larger dynamic factors than the case considering only the gravity load may be required for design. From linear static and nonlinear static analyses, a dynamic factor of 2.0 was obtained as an upper bound. However, it would not be prudent to reduce the dynamic load factors for deficient or robust structures. The effects of falling floors or debris were not considered, and, for deficient structures, damage may be more extensive than was assumed in this study.

- **Effects of structural deficiencies (poor detailing)**

Insufficient development length at connection regions

When a critical column was removed, structures with deficiencies had difficulty in transferring applied loads to alternate load paths due to lack of ductility. If sufficient rotational capacities were provided by adjacent components, they were able to carry load transferred from deficient components. Progressive collapse in a robust structure was not observed when a column was removed. Although plastic hinges in a robust structure were developed, hinges in beams and columns had sufficient rotational capacities to carry additional loads, and alternate load paths were well established.

For a deficient structure, removal of a corner and edge column did not generate progressive collapse due to sufficient resistance and alternate load paths by adjacent members. Removal of an interior column in a deficient structure resulted in a very large rotation demand at an exterior column and may cause propagation of collapse.

By comparing a robust structure and deficient structure, the importance of structural integrity was demonstrated. A robust with good detailing exhibited more ductile behavior and larger load-carrying capacity than a deficient structure as expected.

Shear and Lap Splice Failure

Shear failure in the middle of beams caused by load-redistribution was not observed in the structures analyzed in this study. Shear demand reached 20-50% of the concrete shear capacity.

Lap splice failure at a column immediately above the removed column was not expected even though only 60% of the tensile lap splice length was provided.

Punching Shear Failure

When an edge or interior column in a flat plate structure is removed, punching shear failure at an interior column located closest to the removed column may occur. However, in the flat plate structure analyzed, shear capacity at the adjacent interior column was not exceeded. If longitudinal reinforcement passing through the critical shear perimeter is sufficient to develop membrane action, the possibility of punching shear failure can be reduced.

- **Modification of response**

  Costs associated with providing robust structures may be minimal compared with the costs of improving the performance of a deficient structure. Therefore, code requirements for structural integrity and detailing of reinforcement are an excellent way to reduce the risk of progressive collapse.

8.3 FUTURE RESEARCH NEEDS

Possible extensions of the research include:

- Nonlinear static and dynamic analyses have been investigated for a concrete frame and flat plate structures with deficiencies. Experimental data for frame structures with deficiencies are needed so that comparisons with analytical results can be made.

- In this study, only damage of a column at the first floor of the structure was considered. If any damage above the first floor occurs, debris loads and pounding effects can cause damage on the lower floors and may lead to progressive collapse. These unexpected loads should be studied for application of these loads on a floor system. Possibility of the punching shear failure resulted from the debris loads and pounding effects should be investigated.

- Experimental studies are needed f retrofit schemes to develop alternate load paths or to improve the local resistance of an element subjected to the large deformations that occur when a column is removed.

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